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PAPERS

Paper No. 95

STRUCTURAL TIMBER IN THE UNITED STATES.

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INTRODUCTION.

This paper will attempt a survey of the timber resources of the United States, with reference primarily to structural uses. Information regarding the species of particular interest to engineers is presented, together with a summary of the data obtained by the United States Forest Service on their mechanical properties and structural values. The practical importance of grading rules and commercial specifications for structural timbers justifies a brief reference to the present situation in that regard.

The total amount of standing timber* in the United States suitable for the manufacture of lumber is estimated to be about 2,870,000,000,000 board feet. The original stand has been estimated at 5,200,000,000,000 board feet. Of the difference, it is probable that about one-third has been destroyed by forest fires, one-third lumbered, and one-third wasted.

The forest regions of the United States are shown in Figure 1. Slightly more than one-half of the supply of timber is

*The estimates of stand are taken largely from a report of the Bureau of Corporations of the Department of Commerce on ''The Lumber Industry'', and the Report of the National Conservation Commission. These estimates include, for the most part, only timber of sawlog size, as determined by current requirements. A figure representing the total forest resources of the United States, including fuel, pulpwood, etc., would be materially larger.

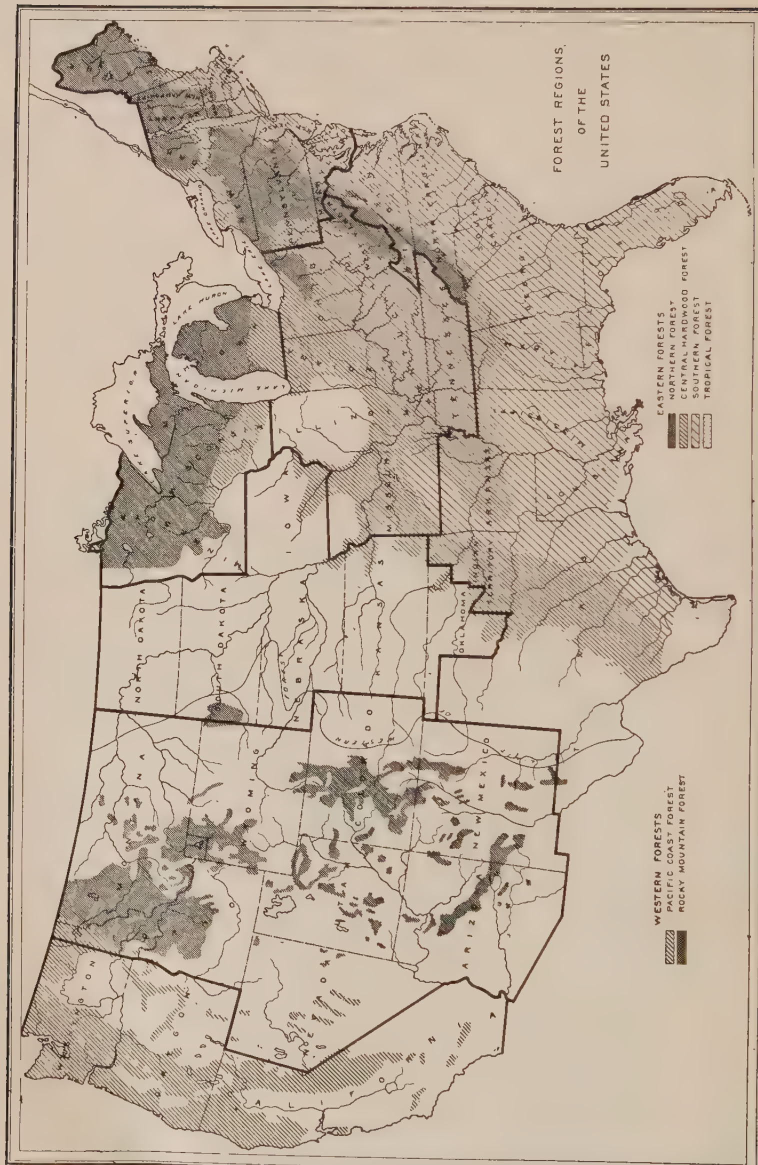

FOREST REGIONS OF THE UNITED STATES
EASTERN FORESTS
NORTHERN FOREST
CENTRAL HARDWOOD FOREST
SOUTHERN FOREST
TROPICAL FOREST
WESTERN FORESTS
PACIFIC COAST FOREST
ROCKY MOUNTAIN FOREST

Fig. 1.

in the Pacific Northwest (Washington, Oregon, Idaho, and northern California); about one-fourth of the total amount is in the Southeast; the remaining one-fourth is distributed throughout the former centers of production, the Lake States and the Northeastern States.

Figure 2 shows the estimated timber supply by States and species.

The amount of lumber handled by the saw mills of the United States, in 1911, 1912, and 1913, by States and species, is shown in Table 1 of the Appendix. The average amount of wood used annually in the United States for the last few years has been about 52,000,000,000 feet B. M. This amount is made up of sawed lumber, ties, mine timbers, wood used for paper pulp, distillation, etc., but does not include fuel, fence posts, and rails. It is distributed among the principal wood-using industries as shown in Table 2 of the Appendix.

The material in Table 2 utilized, at least in part, for engineering construction, is included under (1) Lumber and Structural Timber, (4) Ties, (5) Mine Timbers, (7) Car Construction, (22) Poles, and (23) Ship and Boat Building, or a total of 44 percent of the entire wood consumption. Just what proportion of this 44 percent is used for engineering construction, statistics do not show, but it seems safe to assume that approximately one-fifth of the timber annually consumed in the United States, with a valuation estimated at $100,000,000 is devoted to such uses.

FUTURE TIMBER SUPPLY OF THE UNITED STATES.

About 23 percent of the standing timber in the United States, or 673 billion feet, is in public ownership. The most important of the public timber lands are the National Forests, which aggregate 165 million acres; the timbered portions of the Indian Reservation, which total 7 million acres; and Forest Reserves in the ownership of various States, which exceed upwards of 3½ million acres. On these areas, broadly speaking, the forests are being cut conservatively and extended by artificial planting; protection from forest fires is afforded and a continuous production of wood under approved forestry methods is assured.

Aside from these public holdings, however, there is, as yet, little prospect of the scientific renewal of the present enormous forest resources of the United States. Timber has not yet attained a sufficient value to justify the practice of forestry. Lumbering is still largely the exploitation of virgin timberlands by destructive methods and without regard for the future production of wood, a condition forced upon the industry by its unstable market and the fluctuating, often excessively low, value of its product. Exceptions to this general condition are restricted to small parts of the eastern States, where the practice of forestry is made possible, economically, by exceptionally favorable markets for varied wood products.

The manufacture of lumber in the United States is therefore a nomadic industry, shifting from section to section as the supplies of virgin timber are exhausted. Until 1860, the bulk of the lumber produced was cut from the Alleghany Mountains and northeastern States. For the next thirty years, the vast white and Norway pine stands of the Lake States furnished most of the country's lumber. The industry then, as represented by the greater part of its mill capacity and output, moved into the southern yellow pine belt on the Atlantic and Gulf Seaboard. The production from that section is now at its height, approximating 17 billion feet annually. The total stand of southern yellow pine is estimated at 384 billion feet; hence, at the present rate of cutting, twenty years will witness its exhaustion as a large factor in the American lumber trade. In the meantime, the output of the Pacific Coast, already reaching 8 billion feet of Douglas fir in Oregon and Washington, is steadily climbing; and it is simply a question of years before it will exceed that of the lumber regions of the East.

The shifting character of the lumbering industry in the United States is seen in the change from period to period in the species of timber making up the bulk of its commercial product. Southern pine is now in very general use as a structural timber, but the movement of the industry to the Pacific Coast is going, in the future, to replace it to a considerable extent with Douglas fir, just as the yellow pine has replaced Norway pine, spruce, and various hardwoods in many structural uses.

It is roughly estimated that the total annual drain upon the forests of the United States from fires, insects and lumbering combined, including wood for all uses, is around 100 billion board feet. The annual growth of wood in the forests, most of which is a matter of chance rather than of management or scientific method, is probably not over one-third of this amount. The country is thus evidently drawing upon its forest capital at the rate of 60 to 70 billion feet annually. Long before the present enormous forest resources are eaten up, however, it is more than probable that readjusting economic conditions, particularly higher values for timber and a lower per capita consumption, will reduce the annual drain to an amount not more than the annual growth. With its enormous areas of non-agricultural lands,—apparently best suited to the production of timber, in the natural order of things,—it is probable that the United States will not only supply its own needs but continue to be a large exporter of timber. Even the quality of the lumber placed upon the market, which is of primary interest to the structural engineer, will change but very gradually as old sources of supply renew their stands of merchantable material. For the next thirty or forty years at least, there is no question as to the ability of the United States to furnish high-grade structural timbers from its southern pine and Douglas fir forests, to meet almost any demand for such material from any portion of the world.

PRINCIPAL STRUCTURAL SPECIES.

The principal species used for structural or engineering purposes in the United States fall naturally into three groups:

1 Timbers of the Southeast
2 Timbers of the Pacific Coast
3 Timbers of the New England and Lake States

TIMBERS OF THE SOUTHEAST.

The Pines.

For the past 20 years the southern pines have furnished the larger part of the timber used for building purposes in the United States, and still continue to supply over twice as much saw timber as their nearest competitor, Douglas fir. (See

Table 1.) These pines, including longleaf, shortleaf, loblolly, and other varieties of minor importance, such as Cuban pine, spruce pine, and pond pine, grow principally in a belt from 200 to 400 miles wide along the Atlantic Coast and the Gulf of Mexico, from Virginia to Texas.

Nomenclature.—Considerable confusion exists in regard to the names of the southern pines. Specifications of engineers, architects, and builders often call for southern pine, and allow any or all of the pines in question. A list of the botanical, and some of the common, names of the three principal southern pines is given in Table 4 of the Appendix.

Commercially, the southern pines are often divided into two classes, called "Select Structural" and "No. 1 Structural", or "Select" and "Merchantable", or "Grade I" and "Grade II", or "Longleaf" and "Shortleaf", etc. These class names are descriptive of quality; Grade I material is intended to include heavy sticks with few defects, without reference to their botanical classification; and Grade II material, to include timber of an intermediate quality. Most of the timber classed in Grade I is true longleaf, although clear, heavy sticks of shortleaf and loblolly not infrequently fall in this grade.

The botanical characteristics which will enable the three principal southern pines to be distinguished in the field are given in Table 5 of the Appendix.

Distribution.—Longleaf, shortleaf, and loblolly pine trees do not, as a rule, exceed 30 inches in diameter and 100 feet in height. Logs cut from the southern pines average below 20 inches in diameter at the top.

Longleaf pine commonly grows in pure stands of wide extent, to the almost complete exclusion of other species. It occurs principally in a belt 200 miles wide in the lower part of the Southern States which border upon the Atlantic Ocean and the Gulf of Mexico. Longleaf attains a diameter of 16 inches breast-high, with a height of about 100 feet, in from one hundred and ten to one hundred and forty years.

Shortleaf pine is a tree of the plains and foothills; in the South it rarely grows above an altitude of 2,500 feet, and in the northern portion of its range, not above 1,000 feet. East of the Mississippi River the tree appears scattered among the

hardwoods; while west of the Mississippi it forms large forests on the table-lands of the hill country, especially in Arkansas, northern Louisiana, and Missouri. Under favorable conditions, shortleaf reaches a diameter of 16 inches breast-high and a height of about 85 feet at an average age of one hundred and ten years.

Loblolly pine has taken a most prominent part in the renewal of the forests of the southern Atlantic States, on lands which were once cleared. It grows rapidly when young and

Fig. 3. Longleaf Pine (Pinus Palustris).

does not demand so much direct sunlight as the other southern pines. East of the Mississippi, loblolly pine frequents the low pine barrens and their swampy borders, and farther back from the coast is found on the table-lands of northern Mississippi, Alabama, Georgia, and South Carolina. West of the Mississippi there are heavily timbered areas of loblolly in Arkansas, Louisiana, and Texas as far as the Colorado River. Loblolly pine reaches a diameter of 16 inches breast-high and a height of about 90 feet in from forty to ninety-five years. Trees twenty

years old average about 45 feet in height and from 4 to 6 inches in diameter breast-high.

The ranges of the three principal southern pines are shown on diagram maps in Figures 3, 4 and 5.

Characteristics of Wood.—The wood of all the southern pines is much alike in appearance. The sapwood and heartwood are distinctly marked, the sapwood being yellowish white

Fig. 4. Shortleaf Pine (Pinus Echinata).

and the heartwood reddish brown. The heartwood of old logs is generally heavier than the sapwood on account of more summerwood being formed when the tree was comparatively vigorous. The layers of growth, or annual rings, show distinctly in the wood of these pines and the width of the annual rings generally varies with the age period of the tree, being greatest in early life and least in the sapwood of mature trees. The alternating bands of dark summerwood and lighter colored spring-

wood are distinct in the southern pines. The physical characteristics of the three principal southern pines are given in Table 6 of the Appendix.

Utilization.—Longleaf pine is an excellent structural timber, and finds a wide use in bridge, trestle, warehouse, and factory construction in the form of dimension timbers, posts, piles, and joists. In the building of railroad cars, longleaf has been

Fig. 5. Loblolly Pine (Pinus Taeda).

largely used on account of its strength and stiffness. It is also employed to a large extent for flooring, on account of its hardness and wearing qualities. Longleaf furnishes a very satisfactory material for paving blocks, and in the United States is largely used where wood is employed as a paving material.

Shortleaf and loblolly are used principally for building lumber, such as interior finish, flooring, ceiling, frames and sashes, wainscoting, weatherboarding, joists, lath, and shingles;

for boxes and packages, in the form of boards and veneers, and in cooperage for the manufacture of slack barrels. Shortleaf and loblolly are also used to some extent for construction purposes, in bridge and trestle work, and heavy building operations. The use of preservative processes, by which the timber under treatment is impregnated to a certain depth with an antiseptic liquid, usually creosote oil, which prevents or retards decay, has increased the use of shortleaf and loblolly for structural purposes.

Many of the mills manufacturing southern pine lumber are located near the coast, where their wharves are directly accessible to lumber schooners and, oftentimes, can be reached by ocean steamers. In this way a large amount of lumber is shipped to the northern markets and also abroad. The inland mills ship north or west by rail, or to the nearest coast shipping point for foreign trade. The principal northern coast markets, such as Baltimore, Philadelphia, and New York, are supplied with southern pine rough lumber almost entirely by cargo shipments from Norfolk, Charleston, Savannah, Jacksonville, Pensacola, Mobile, and New Orleans. Dressed stock is shipped by rail, and the railroads supply all southern pine lumber to the large northern inland markets, such as Chicago, Buffalo, Cincinnati, Pittsburgh, St. Louis, and Kansas City. These shipments are collected from various points convenient to localities where mills happen to be operating. Port Arthur and Galveston are also important lumber markets, especially as shipping points to foreign countries.

Cypress.

The amount of standing cypress *(Taxodium distichum)* in the United States is estimated at about 40,000,000,000 feet. The cut has been steadily increasing and in 1913 was slightly over 1,000,000,000 feet. On account of the slow growth of cypress and its scanty reproduction, the prospects for a long continued supply of this wood are not bright.

The name, bald cypress, is sometimes used, due to the tree losing its foliage in winter. The terms, white, red, yellow, and black cypress are frequently employed to indicate some special or desirable property in the wood from certain localities, and have led to much confusion.

Cypress grows in commercial quantities along the Atlantic Coast and Gulf of Mexico from Maryland to Texas, and in the Mississippi Valley as far north as the junction of the Mississippi and Ohio Rivers. Throughout its range, it occupies parts of swamps and creek bottoms and the deltas of rivers. The geographical range of cypress is shown in Fig. 6.

On fertile lands, cypress attains large dimensions, generally a height of over 100 feet and a diameter of more than 4

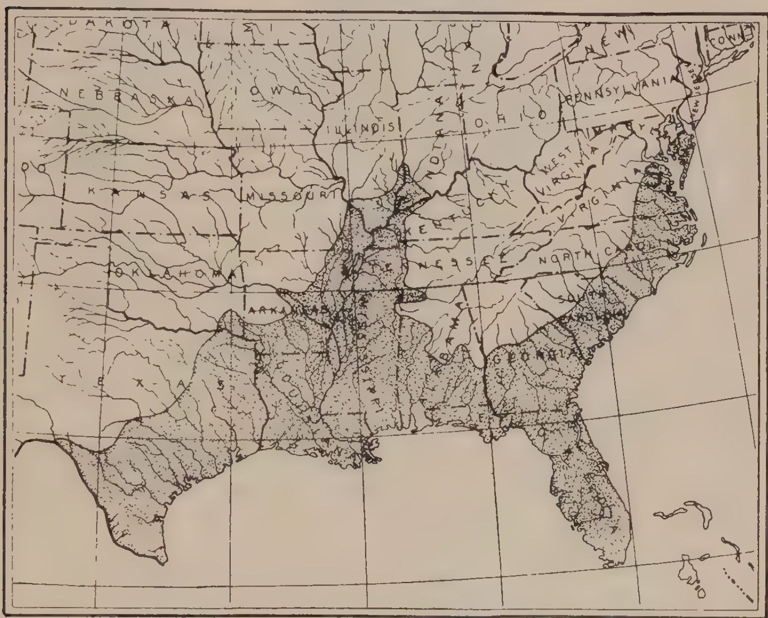


Fig. 6. Bald Cypress (Taxodium Distiehum).

feet above the swell or bottle butt. An age of 500 years is not at all uncommon. On poor soils, the trees are much smaller.

The wood of cypress is rather dark in color, fine-grained, and with narrow rings (slow growth) sharply defined. The sapwood is from ½ to 4 inches wide and lighter in color than the heartwood. The heartwood is especially noted for its durability.

Cypress is cheaper than high grade white pine and dearer than high grade southern yellow pine. It can be had in large dimensions free of knots and defects.

It is not so easily kiln-dried as the pines and requires a longer drying period.

Cypress is used largely for sash and door work and exterior house construction. Also for tanks and silos, and in fact for all purposes where it is exposed to the weather or to the action of water.

TIMBERS OF THE PACIFIC COAST.

Douglas Fir.

Douglas fir *(Pseudotsuga taxifolia)* is one of the most important of American woods. In point of production it ranks second to the southern pines, which, however, are made up of several species. Estimates place the available supply at over 500 billion feet board measure—a much greater amount than for any other species grown in the United States. The center of lumber production, which is now in the Southeast, will doubtless shift to the Northwest as the southern pine forests become more and more depleted, making Douglas fir the principal lumber tree of the country.

Douglas fir is known by various names in different sections of the country. The most common names and the States in which they are used are:

Douglas fir (Utah, Oregon, Colorado, Washington).
Red fir (Oregon, Washington, Idaho, Utah, Montana, Colorado).
Douglas spruce (California, Colorado, Montana).
Yellow fir (Oregon, Montana, Idaho, Washington).
Spruce (Montana, California).
Oregon pine (California, Washington, Oregon).
Fir (Montana).
Puget Sound pine (Washington).

The name Douglas fir has been adopted by the Forest Service and by various trade and technical associations, and is coming into general commercial use.

The geographical range of Douglas fir is shown in Figure 7. Douglas fir is most abundant and attains its largest size not far above sea level in southern British Columbia and in the region between the coast of Washington and Oregon and the

western foothills of the Cascade Mountains. Here, large trees crowded close together reach a height of from 200 to 300 feet, forming, either alone or mixed with western hemlock, very

Fig. 7. Douglas Fir (Pseudotsuga Taxifolia).

dense forests that yield from 35,000 to 60,000 board feet per acre, and sometimes as much as 100,000, and in one recorded instance, 500,000* feet.

* Transactions of the Royal English Arboricultural Soc., Vol. VI, p. 228.

In the eastern part of its range in the Rocky Mountain region, where the rainfall is not abundant and where extremes of climate occur, the trees are much smaller, rarely over 1½ feet in diameter and 90 feet high. In this section the stand ranges from 2000 to 8000 board feet per acre.

The spring and summerwood in the annual rings vary greatly in density. The spring growth is soft and spongy and almost white, while the summerwood is hard and flinty and very dark. The number of rings per inch may vary from as few as four or five to as many as forty-five.

Douglas fir is extensively used in the building trades; by the railroads in the form of ties, piling, car and bridge material; and by many of the manufacturing industries of the country. It is an excellent structural material and is probably most widely known in this capacity. The large, straight, clear trunks of Douglas fir make it possible to supply very large timbers of high quality. Pieces 2 feet by 2 feet in section and 100 feet long can readily be supplied by mills equipped to handle such lengths.

Douglas fir is shipped all over the United States, except to the Southeast; and in the Mississippi Valley and eastward comes into sharp competition with southern yellow pine. Large amounts are exported and the Panama Canal will probably extend the markets for this species still further.

Western Hemlock.

Western hemlock *(Tsuga heterophylla)* has often been considered as an inferior wood, especially in localities where it is not well known. The results of recent investigations of its properties show this prejudice to be unfounded and that it deserves a place as an important western wood.

Western hemlock is known under a variety of names in different localities. It is often given a fictitious name for the purpose of assisting in marketing the lumber. Certain commercial organizations, however, are beginning to place hemlock on the market under its own name. The common names in use are hemlock, western hemlock, hemlock spruce, western hemlock spruce, western hemlock fir, Prince Albert fir, gray fir, silver fir, and Alaska pine. The names gray fir and Alaska pine are favorite ones among lumbermen of the Pacific Northwest,

while in England, western hemlock fir and Prince Albert fir are used.

Western hemlock is found along the Pacific Coast from Alaska to northern California, and as far inland as British Columbia, northern Idaho and northwestern Montana. See Fig. 8. The best stands are found in the coast region and through the Cascade Mountains at an elevation of from 1500 to 3500 feet. In

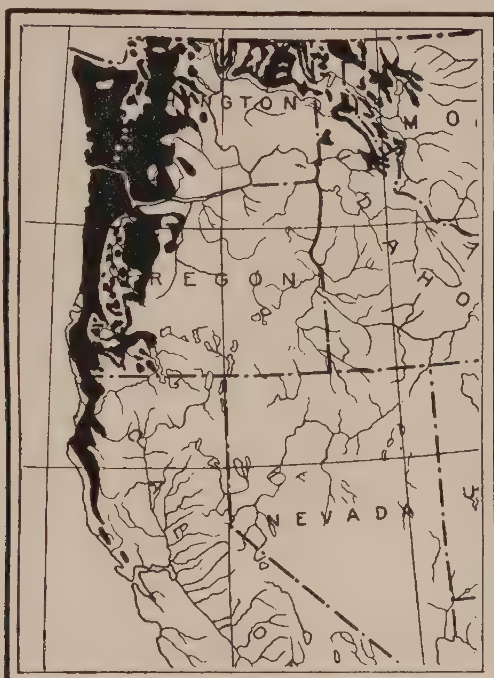


Fig. 8. Western Hemlock (Tsuga Heterophylla).

the State of Washington western hemlock has been estimated to form approximately 13 percent of the merchantable timber.

Mature trees reach a diameter of from 2 to 5 feet and attain a height of from 125 to 150 feet. Exceptional trees have measured 8 feet in diameter and 250 feet in height.

The heartwood of western hemlock is almost white. The yellowish white sapwood forms a very small proportion of the trunk and is generally not over one inch in thickness. The

green wood has a distinctly sour odor, which is not present in seasoned material. The lumber is often mixed with Douglas fir and sold and used for the same purposes.

Western hemlock is suitable for all but the heaviest construction work. It has a limited use in bridge and trestle building and has been employed in caisson construction. In house construction it is used a great deal as a framing material. For this class of work it is satisfactory, and, locally, brings the

Fig. 9. Redwood (Sequoia Sempervirens).

same prices as Douglas fir. Western hemlock dimension stock in cargo shipments commands a lower price than Douglas fir.

Redwood.

Redwood *(Sequoia sempervirens)* grows on the California Coast in a strip extending inland from 10 to 30 miles and extending from the northern border of the State southward to a little below Monterey.

Redwood is sometimes confused with the "bigtree" *(Sequoia washingtoniana)*, a timber of similar color but softer in texture, noted for its great size and age. The "bigtree",

however, is logged only in small quantities. Mature redwood trees attain an age of from 500 to 800 years, much less than the age reached by the "bigtree". The average trunk is from 6 to 10 feet in diameter. Trees 20 feet in diameter and 300 feet or more in height have been measured.

The heartwood varies in color from a light cherry to a dark mahogany. The narrow sapwood is almost white. The number of annual rings on a cross-section of redwood may vary from 10 to 50 in an inch. The bark is reddish brown and very fibrous.

Redwood is used for all kinds of construction and finishing purposes, for ties, shingles, paving blocks, telegraph poles, and tank and pipe staves. The lasting qualities of redwood make it especially suitable for building construction when the conditions are favorable to decay.

Some 60 or 70 California mills turn out the entire cut of redwood. The principal distributing points for redwood are San Francisco and Eureka, for both rail and water shipments. In addition to the redwood lumber distributed throughout the United States, large quantities are exported to Australia, South and Central America, Mexico, Europe, Africa, and the Hawaiian Islands.

Western Larch.

Western larch *(Larix occidentalis)* is, at present, but little used as a structural material and knowledge of its properties is limited. The wood is more commonly known as larch or tamarack in the region of its growth.

Western larch grows throughout the basin of the Columbia River from southern British Columbia and the western slope of the Continental Divide in Montana to the eastern slope of the Cascade Mountains in Oregon. The best stands of larch timber are found in northern Idaho, northeastern Washington, and northwestern Montana. Western larch generally occurs mixed with other species. Its range is shown in Fig. 10.

The annual rings show a distinct dark and light-colored band. In material tested by the Forest Service the number of rings per radial inch varied from 11 to 60. The heartwood is reddish brown in color and the sapwood yellowish white. The latter runs from one-half to one and one-half inches in thick-

ness for trees up to 3 feet in diameter. Knots are generally sound and not over one and one-half inches in diameter. They are common and frequently occur in groups or clusters. Western larch is subject to ring shakes and trees are frequently butted to get rid of such material.

Western larch is a hard, heavy wood, having many of the qualities of Douglas fir and is often found in the same uses. It is used to some extent for cross-ties and has given satisfaction. Its hardness, fine grain and pleasing reddish color are

Fig. 10. Western Larch (Larix Occidentalis).

making it more and more popular for flooring and other forms of finish. The chief use of western larch as a structural wood is in local railroad and mining centers, and for framing in the Northwestern States. Larch is not as strong as Douglas fir, and, considering also its limited supply, will have no such importance as fir as a structural timber. As the grading of structural woods becomes more refined, on the basis of fundamental qualities, larch will undoubtedly have its markets extended.

Spokane is the principal distributing point for the comparatively small amount of western larch that is shipped for consumption outside the region of growth.

Western Yellow Pine.

The wide range and large supply of western yellow pine *(Pinus ponderosa)*, and the fact that in extensive regions it is

Fig. 11. Western Yellow Pine (Pinus Ponderosa).

practically the only timber, give it a very important place as a structural timber.

The total stand of western yellow pine in the United States, in 1909, was estimated at 275 billion feet B. M. Douglas fir was the only species that was given a greater total. The stand of the three southern pines taken together was placed at one-third more than western yellow pine.

The names under which western yellow pine goes vary with

the locality; among them are bull pine, red pine, pitch pine, western pitch pine, western pine, and California white pine.

Few trees have a range as wide as that of western yellow pine. It has its best development on the Pacific Coast, but it covers one-third of the United States. It is cut in Arizona, California, Colorado, Idaho, Montana, Nevada, New Mexico, Oregon, South Dakota, Utah, Washington, and Wyoming.

Mature trees reach a diameter of from three to seven feet and a height of from 100 to 200 feet. The heartwood is reddish white in color, and the thick sapwood is almost white. The wood of western yellow pine varies widely in resin content, rate of growth, and color, due in part to the wide range in climate, altitude, and soil over which the species grows.

Western yellow pine has furnished a large part of the ties and bridge and trestle timbers for railroad building in the western mountains and plateaus. Its uses range from the coarsest construction to the making of patterns, as a competitor of white pine. It is used largely in house construction, for flumes, as a mine timber, and for bridge work. The lumber is widely exported, and reaches New Zealand, Australia, England, Ireland, Scotland, the continent of Europe, and elsewhere. Common grades of lumber are shipped as far east as the Lake States, and the higher grades to the Atlantic Seaboard.

Western yellow pine is a softer and lighter wood than Douglas fir or longleaf southern pine. Western yellow pine is not primarily a structural timber, but one more adapted to shop uses, planing mill products and finish; and because of its soft texture, white color and non-resinous character, is marketed largely as western white pine.

TIMBERS OF NEW ENGLAND AND THE LAKE STATES.

Norway Pine.

Norway pine *(Pinus resinosa)* was formerly much used in construction work, but as the northern forests have become more and more depleted it has been replaced by wood from the southern forests and the northwest. The species is still used to some extent, however, in the Lake States for structural purposes.

Norway pine is also known as red pine, hard pine, and Canadian red pine. Its commercial range, as shown in Fig. 12, lies in Michigan, Minnesota, and Wisconsin, and in the provinces of Canada. A small quantity is cut in New York, Pennsylvania, and New England.

The tree attains a diameter of from 2 to 3 feet and a height of from 75 to 125 feet. The wood has a rather coarse grain, and is somewhat resinous, lying between white pine and pitch pine in this respect. In color the heartwood is light red and the sapwood yellow or often almost white.

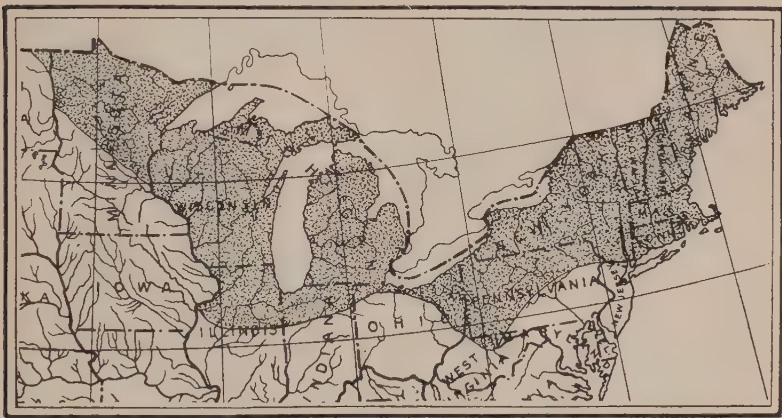


Fig. 12. Norway Pine (Pinus Resinosa).

The wood was largely used, and is still used, both in this country and in Canada and England, for masts, spars, piling, and deck plank. It is used in car construction, including sills and frames.

Tamarack.

Tamarack *(Larix laricina)* is at present a structural timber of minor importance and is used only locally in the Northern States. It is known also as larch, hackmatack, American larch, black larch, and red larch.

Tamarack has the widest range of the northeastern conifers. It has its approximate southern limit in northern Pennsylvania, Indiana, and Illinois, and extends northward to the Arctic circle and westward, in the United States, to central

Minnesota. The best specimens grow, where the moisture is not excessive, on the edge of swamps and along the banks of sluggish streams.

The average diameter of tamarack is from 12 to 20 inches and the average height 60 feet. Forest-grown trees have a clean, straight bole. The wood resembles Norway pine. The grain is rather coarse. Lumbermen recognize two varieties of tamarack, the red and the white, the distinction being based on the color of the heartwood, which varies with the soil and climate.

The wood is used in ship-building, sometimes in the form

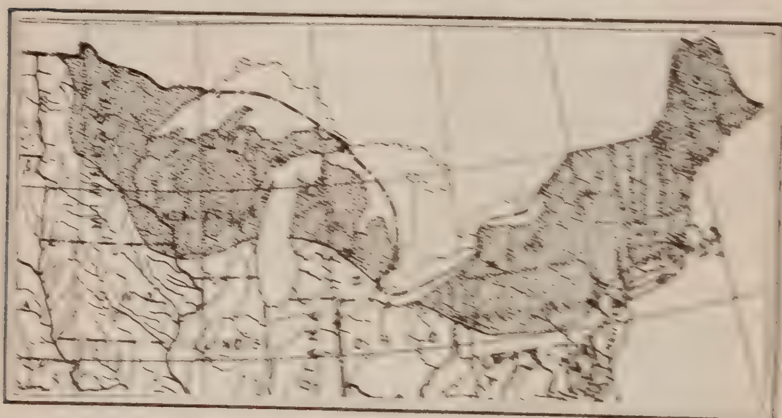


Fig. 13. Tamarack (Larix Laricina).

of knees, and also for spars and masts, and for ties, and telegraph poles.

Red and White Spruce.

The wood of red and white spruce (*Picea rubens* and *Picea canadensis*) is very similar in appearance and properties, and the two are commonly considered simply as "spruce", as far as their use for structural purposes is concerned.

Red spruce has a reddish-brown bark, and white spruce, a light grayish bark. In both, the needle-shaped leaves are set thickly on the branches. The cones of white spruce are about 2 inches long and those of red spruce slightly shorter.

Red spruce grows in New England, Michigan, Wisconsin, Minnesota, North and South Dakota and Montana, and practi-

cally throughout the whole of Canada and Alaska, except near the west coast.

White spruce has a more limited range. It extends from New England, through New York, Pennsylvania, and West Virginia, to eastern Tennessee.

The wood of both spruces is dull white, sometimes tinged

Fig. 14. Red Spruce (Picea Rubens).

with yellow or red. The sapwood is not distinctly marked. The summerwood rings are more clearly marked in the red spruce.

Red and white spruce are used in New England, and to a less extent elsewhere, for joists and small forms of structural timber. These species have a high value for paper pulp and will, doubtless, never be of more than local importance as structural material.

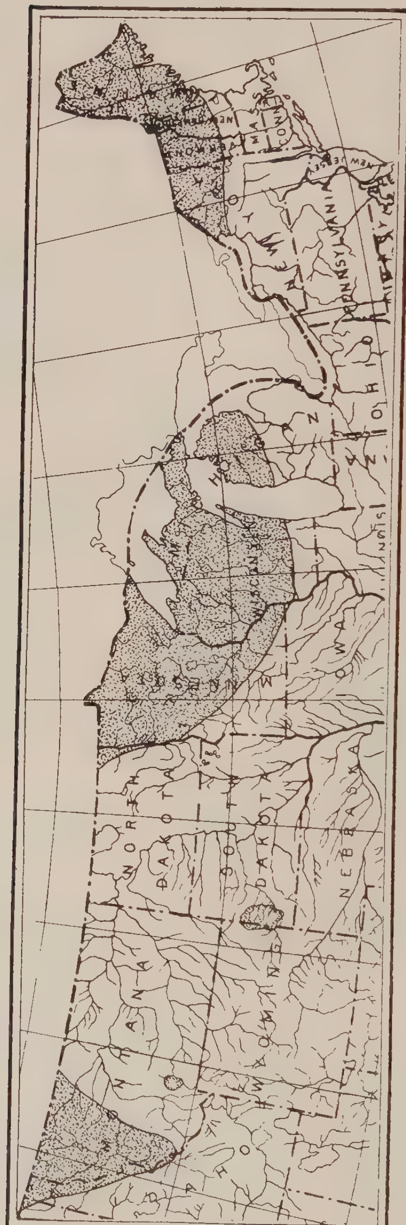


Fig. 15. White Spruce (Picea canadensis).

Eastern Hemlock.

The output of hemlock *(Tsuga canadensis)* has been declining for several years, due to the lessening supply of the timber. The cut in 1912 was 29 percent less than in 1899. The tree is sometimes called "hemlock spruce", and sometimes simply "spruce", in Pennsylvania, Delaware and the southern parts of its range.

The commercial stands of eastern hemlock are found, principally, in Wisconsin, Michigan, West Virginia, Pennsylvania, New York, and New England. It has been estimated that Wisconsin and Michigan contain half of the present supply.

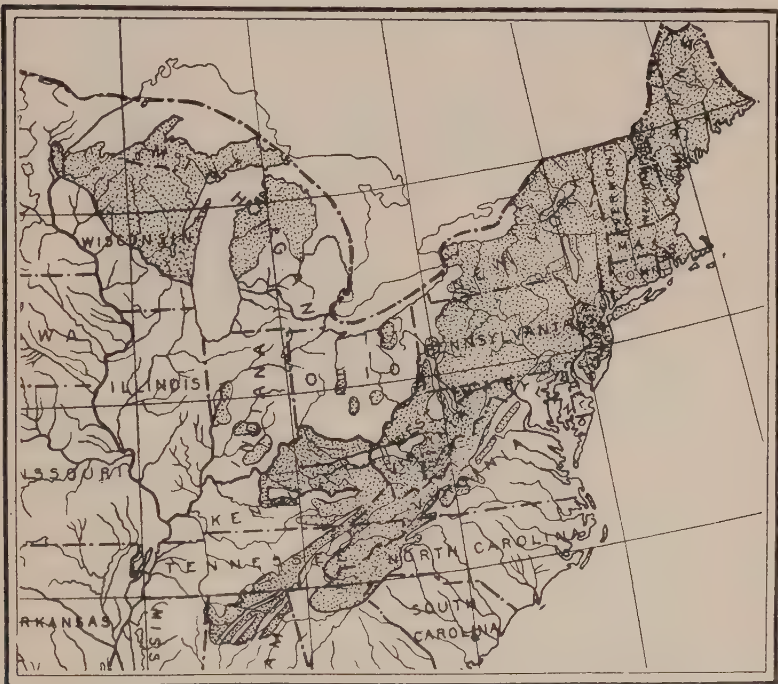


Fig. 16. Eastern Hemlock (Tsuga canadensis).

The wood is reddish brown in color. The sapwood is difficult to distinguish from the heartwood. The annual rings are distinct.

About two thirds of the hemlock timber cut in the United States is used in its rough form for joists, rafters, boxes, con-

crete forms, construction lumber, etc. The remainder is further manufactured into finished products before reaching the market.

The markets recognize Lake States hemlock and Eastern States hemlock. Generally the two classes do not go on the same markets.

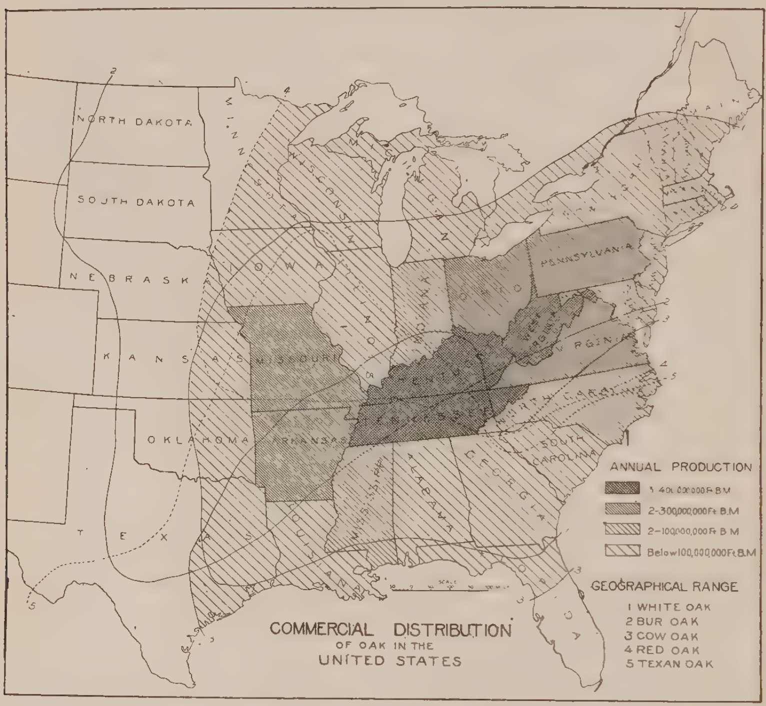


Fig. 17. **Oak.** (Based on Lumber Cut of 1912.)

Oak.

Oak is still used in large amounts for construction purposes, especially railroad ties, although the increasing cost of the timber in dimension sizes, due to its special fitness for veneer, furniture, and finish, is making the use of oak as a construction timber more and more local.

Over 50 different species of oak grow in the United States. The range of the principal commercial species is shown in Figure 17. The largest stands are located in the southern Appa-

lachian region. Tennessee, Kentucky, and West Virginia were the largest contributors of oak timber in 1912.

The wood of the oaks is generally considered, commercially, as either "white oak" or "red oak". White oak generally includes the species with comparatively dense wood and light color and with the pores in the wood closed; and red oak, the species of lighter weight and, frequently, reddish color and with numerous open pores, which render the wood unsuitable for the manufacture of liquid containers and make it more susceptible to decay.

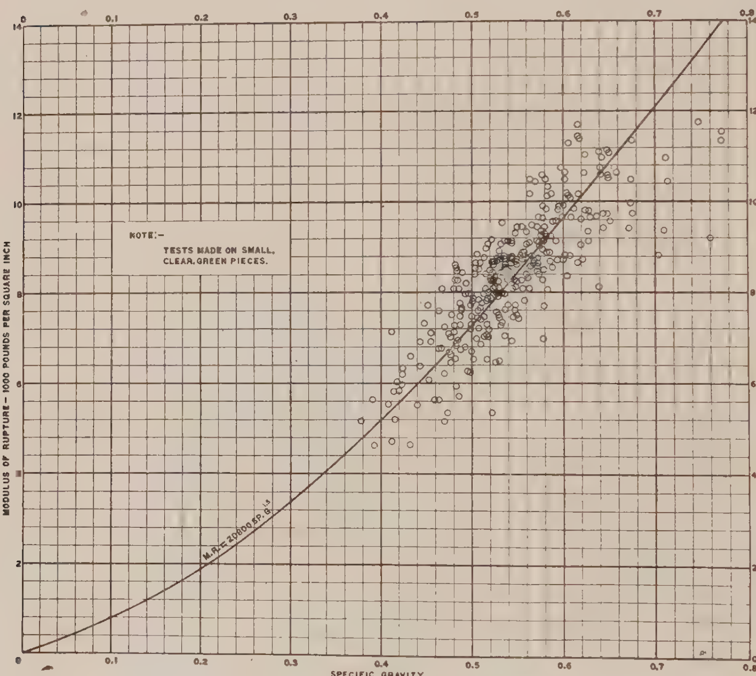
Oak is used largely in vehicle construction and for framing where strength is required. In the regions of its growth, it is employed in bridge construction as stringers, posts, and flooring, for piling, and for car sills and bolsters.

FACTORS AFFECTING THE VALUE OF STRUCTURAL TIMBER.

Until recent years, large quantities of high-grade timber suitable for engineering construction were available in the United States. But little necessity was felt for economy in sizes or careful quality grading. Engineers and builders demanded and received structural timber of the highest quality. There was an ample supply in the forests to cut from, and timber of questionable quality was left. Years of heavy cutting in the finest stands of structural timber, naturally, have had their effect, and in many localities it is now somewhat difficult to obtain shipments of the highest quality material. The practice of cutting timber of an intermediate or poor quality into structural material, as well as high-class timber, and the growing tendency to place these timbers of various qualities on the market has made the question of grading rules and methods of inspection of increasing importance. The purpose of such rules is to separate structural timbers into classes in such a way that the material in any one class will be of approximately uniform quality. Grading rules must be based on test data and a knowledge of the requirements of the trade and the ability of the producer to meet these requirements.

The two qualities of greatest value in the wood of structural timber are strength and durability. In order properly

to select material for a given use, a knowledge of the influence of the various physical characteristics of timber on its strength and durability becomes necessary. The characteristics which may be considered in judging the properties of timber are density of wood, direction of grain, moisture condition, proportion of sapwood, and, in addition, defects, such as knots, checks, and shakes.



RELATION BETWEEN BENDING STRENGTH AND DRY WEIGHT FOR LONGLEAF PINE

Fig. 18.

Density.

The strength, hardness, shock-resisting ability, and stiffness of wood vary with its weight in small, clear, straight-grained pieces of the same moisture content. The relation of bending strength to dry weight for a single species (longleaf pine) is shown in Figure 18. The same relation is shown for 113 different species in Figure 19. From these figures it will be seen that as the weight increases the strength increases, both in the

case of pieces all cut from the same kind of wood and in the case of average values of many different kinds of wood. What, then, is needed is some means of estimating density from a visual inspection. Wood is made up of concentric rings of growth. Each ring consists of two parts, springwood and summerwood. The springwood, as its name indicates, is formed first, and in the woods commonly used for structural purposes is the lighter colored, softer part of the ring. The summerwood is built up later in the season and is darker and heavier than the springwood. (Fig. 20.) In certain woods, the change from springwood to summerwood is distinctly marked, so that the proportion of summerwood in an annual ring or in a cross-section can be closely estimated. (Fig. 21.) Summerwood is considerably heavier than springwood. A number of tests on small pieces of summerwood and springwood cut from wide-ringed loblolly pine showed a density and strength about twice as great in the summerwood as in the springwood. The proportion of summerwood in a cross-section of a stick, therefore, becomes a means of estimating its density and strength. It should be remembered that density is the basic factor governing strength and that the proportion of summerwood as a satisfactory indicator of strength is dependent on the difference in density of the two parts of the annual ring and on the closeness with which these parts can be differentiated.

It is to be regretted that very little data are available in regard to the influence of density on durability in the case of any one species. The heavier species are not necessarily the most durable, since some of the lightest woods, as cedar, redwood, and cypress, have excellent lasting properties, while some of the heavier hardwoods, as hickory and red oak, are notably less decay-resisting. On the other hand, the lighter woods, basswood and cottonwood, are not durable, while the heavy woods, osage orange and locust, are durable.

As to the resin content of the pines being a means of judging their durability, there are apparently little data available on this point also. While the toxic properties of resin appear to be very low, the presence of resin in wood keeps out moisture and air, and in this way it acts as a preservative. It is quite a common occurrence to find lying in the woods pine logs which

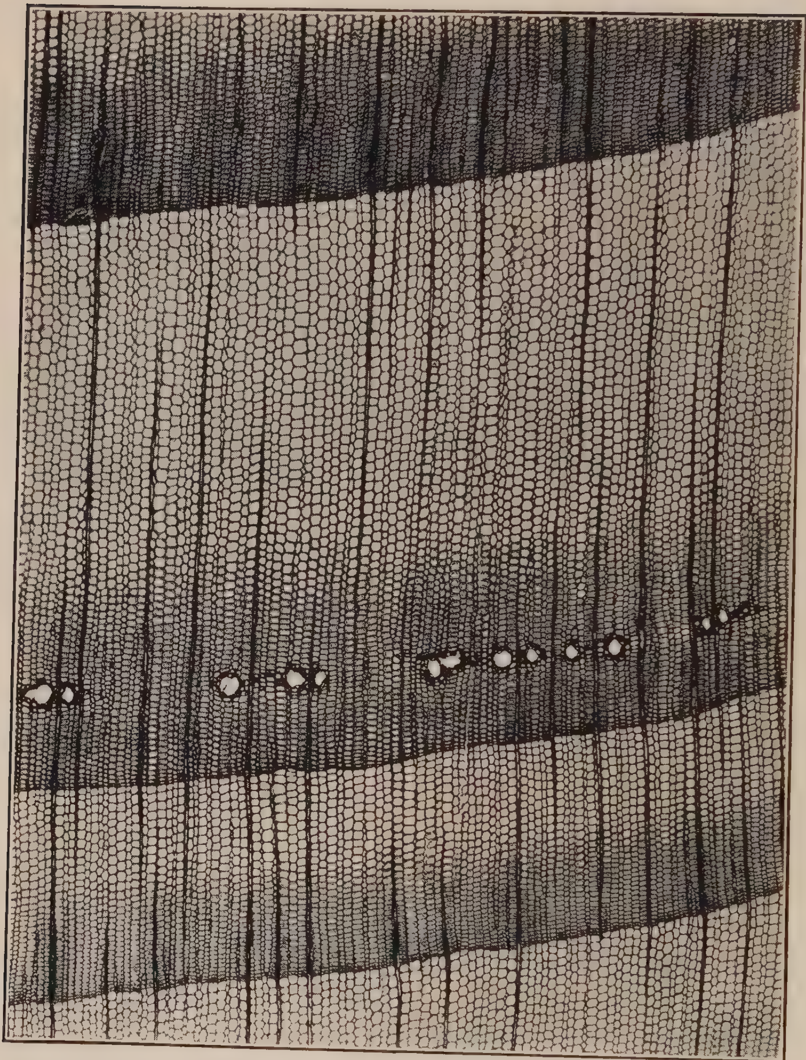


Fig. 20. **Photomicrograph of Douglas Fir.**

Note porous springwood and denser summerwood.

at first sight are apparently in a fair state of preservation, but on examination the sap is found to be entirely decayed, the only portions remaining sound being such parts of the heart as are permeated with resin. It is also a well known fact that

fence posts chosen from pitchy material last longer than similar posts from wood with a small pitch content. It seems reasonable, therefore, to consider that a high percent of resin is favorable to durability.

Fig. 21. Cross Section of Loblolly Pine Beams.
Note different proportions of summerwood.

Direction of Grain.

By grain is commonly meant the lines formed by cutting the rings of annual growth. If a stick is sawed from a log in such a way that these lines run diagonally across the stick instead of parallel to the edges, the load that it will carry as a beam will be considerably reduced, and, moreover, failure will be more complete when it occurs.

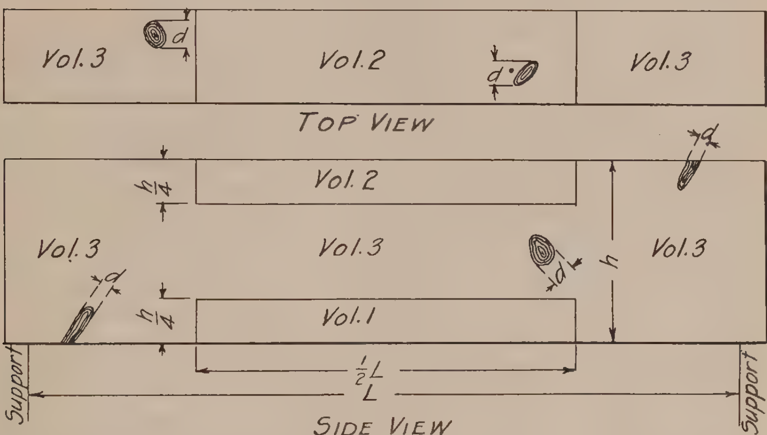
There is, however, another kind of cross-grain, known as spiral grain, which is more difficult to detect than that just described, but which also weakens the timber and may cause complete failure. The wood of the conifers may be considered as built up mainly of elongated cells running lengthwise with the trunk, with a much smaller proportion of cells in small bundles located at right angles to them and lying radially. These groups or bundles of cells are termed pith rays. Their cross-sections appear on a magnified tangential or flat-grained face of timber as figures tapering on each end, with the long axis vertical, and on a radial section as light-colored bands. In oak, as an example, the pith rays are very much in evidence, and form the figures in quartered material. In the conifers, however, the pith rays are not generally noticeable. Not infrequently the cells of which a tree is built up follow a spiral course around the pith instead of lying vertically. In such trees the cross-sections of the pith rays are also inclined or lie diagonally and form an indication of spiral grain. It is quite possible for a piece to be straight-grained, so far as the annual rings are concerned, and still to have a spiral grain. Spiral grain is often indicated in seasoned material by the diagonal direction of fine surface checks which occur at the pith rays. Splitting a block of wood will show whether or not it has spiral grain.

Moisture.

While the effect of moisture on strength is very marked in small clear pieces of wood (Fig. 19), it is much less in evidence in large pieces. Under the present methods of seasoning structural material, any increase in the strength of the wood fiber is commonly offset by the checks and shakes induced in the process, so that the strength of large beams, on the average, is increased little, if any, over that of the green sticks. The moisture condition of structural material from a durability standpoint is, however, an important consideration, especially in cases where the beams are to be placed in a poorly ventilated location. Under such conditions, careful seasoning becomes necessary if even a reasonable life is to be expected, and it is highly probable that preservative treatment will be found desirable. Green timber is also liable to shrinkage or distortion.

Sapwood.

The proportion of sapwood in a stick of timber has no bearing on its strength. If a tree is cut at a time when it is putting on its densest wood, pieces cut from the sapwood will be the stronger. If, on the other hand, it is cut when mature, so that the densest wood has changed to heartwood, then pieces cut from the heartwood will be the stronger. Sapwood is, however, notably less durable than heartwood, and, generally, only a small proportion of sapwood is allowed in the highest class structural timber. If the timber is to be treated with a preservative, the presence of sapwood is not considered so detrimental, as it is easier to treat than heartwood.



Division of beam into Volumes for location of knots.
Measurement of knots on horizontal and vertical face of beam.
d = diameter of knot.

Fig. 22.

Defects.

Knots, checks, and shakes are the most common kind of defects affecting strength. In timber for use as beams, the influence of knots on strength is largely a matter of location. Figure 22 shows a method of dividing a beam into three volumes with reference to the location of knots. Numerous tests have shown that knots occurring in Volume 1, which occupies the lower quarter of the central half of the beam, have considerably

more weakening effect than similar knots occurring in other volumes. Loose or rotten knots are, of course, more harmful than those closely knit with the surrounding wood. A comparatively small knot situated near enough the lower edge of a beam to turn the grain off is more harmful than a larger knot so placed as to allow the grain to be continuous in passing. In some cases, knots near the neutral plane may act as pins and tend to strengthen a beam against failure in longitudinal shear. In a series of tests on loblolly pine beams, those with knots in Volume 1 had about 75 per cent of the strength of sticks with knots in the remaining portions.

Checks are caused by the stresses set up in seasoning. Structural timber in large sizes is difficult to season without more or less checking, even under favorable conditions; frequently it is so exposed as to cause the surface to dry much more rapidly than moisture can be transmitted from the inner portions to the surface, and thus the outer portions dry and shrink, while the center is still wet, and checking results. A shake is a separation between two annual rings; generally the separation occurs in only part of the ring, but sometimes it is complete. Shakes are ascribed to the bending action of wind on the standing tree. They are frequently not visible in green timber, but show up later during seasoning. Both checks and shakes weaken beams in their ability to resist horizontal shear in proportion as they affect the area of the beam near the neutral axis.

In the case of a stringer, 8 x 16 inches in section, tested in bending over a 15-foot span, failure may occur by tension or tearing apart of the fibers in the lower part of the beam, by compression or crushing in the upper part, or by shearing along the neutral axis. The results of shearing tests on small clear pieces show that, under the conditions given, failure would always occur in tension or compression long before the beam would have reached its maximum shearing stress, provided the shear resisting area was intact. In tests on commercial material, however, horizontal shear is a common form of failure, due to the fact that the area that resists shear is frequently weakened by checks and shakes. A comparison of the stresses obtained from tests on 8- by 16-inch by 16-foot stringers and small

clear pieces, shows that the shear-resisting area is frequently reduced about 50 per cent, due to checks and shakes.

Checks and shakes influence the durability of timber by allowing the collection of moisture and the formation of conditions suitable to the propagation of fungus growth. Knots have little effect on durability, one way or the other, except in cases where they become loose and fall out, leaving holes that are apt to collect moisture and produce conditions favorable to decay.

GRADING RULES AND SPECIFICATIONS FOR STRUCTURAL TIMBER.

At the present time, a large number of different rules and specifications for the grading of timber are in use in the United States. Practically all of the lumber associations have adopted rules for the purpose of classifying the lumber manufactured by their members. The railroads also have specifications for the selection of their timber, and a number of engineering societies have from time to time brought out timber specifications or grading rules.

The most efficient grading rule is one that will pass the largest amount of material suitable for the purpose intended and throw out the largest amount of material not suited for such a purpose. Many of the present grading rules are either very general and loose, or else so rigidly drawn as to exclude timbers of high strength. The following are examples of grading specifications, some of which cover only one grade, while others have several grades:

Commercial Specifications.

Specification A.

Specification A is used by one of the leading transcontinental railway systems.

> All timber must be of the best description of the kind required. It must be sawed square and to proper dimensions. It must be free from all loose, large, or unsound knots, sap, sun cracks, shakes, wanes, or other imperfections or defects which would impair its strength or durability.

Specification B.

Specification B is used by the Isthmian Canal Commission.*

> **Stringers.** Douglas fir shall show not less than 85 per cent heart on any face and not less than 70 per cent on any edge; it shall show not less than an average of 12 annual rings to the inch. Sound knots less than 3 inches in diameter shall be permitted in the vertical faces of the stringer at points not less than one-fourth the depth from the edge of the piece.

Specification C.

Specification C is the standard adopted by the Pacific Coast Lumber Manufacturers' Association (now the West Coast Lumber Manufacturers' Association) in 1911. But very slight changes have been made in this set of grading rules since 1899:

> **Clears.** Shall be sound lumber well sawed, one side and two edges free from knots and other defects impairing its use for the probable purpose intended. Will allow in dimensions larger than 6 inches by 10 inches pitch pockets when not extending through the piece; light-colored sap on corners not exceeding 3 inches on face and edge, knots 2 inches and less in diameter, according to size of piece, when on one face and one-half of each corresponding edge, leaving one face and upper half of each edge clear.
>
> **Selects.** Shall be sound, strong lumber, well sawed. Will allow in sizes over 6 inches by 6 inches, knots not to exceed 2 inches in diameter, varying according to the size of the piece; sap on corner not to exceed 2 inches on both face and edge; pitch pockets not to exceed 6 inches in length. Defects in all cases to be considered in connection with the size of the piece and its general quality.
>
> **Merchantable.** This grade shall consist of sound, strong lumber, free from shakes, large, loose, or rotten knots and defects that materially impair its strength, well manufactured, and suitable for good, substantial constructional purposes. Will allow slight variations in sawing, sound knots, pitch pockets, and sap on corners, one-third the width and one-half the thickness or its equivalent. Defects in all cases to be considered in connection with the size of the piece and its general quality. In timber 10 inches by 10 inches and over, sap shall not be considered a defect. Discolorations through exposure to elements, other than black sap, shall not be deemed a defect excluding lumber from this grade, if otherwise conforming to merchantable grade.

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* The Canal has recently adopted the density requirement in the Forest Service rules for southern yellow pine. Seventeen million board feet have been inspected with satisfaction to buyer and seller.

Common. This grade shall consist of lumber having knots, sap, and other defects which exclude it from grading as merchantable, but of a quality suitable for rough kinds of work.

Specification D.

Specification D was prepared by the American Society for Testing Materials* and, with slight changes, was adopted by the American Railway Engineering Association.

Standard specifications for Douglas fir and western hemlock bridge and trestle timbers. To be applied to single sticks and not to composite members:

Standard Heart Grade. Shall include yellow and red Douglas fir and western hemlock. White Douglas fir will not be accepted.

General Requirements. All timber shall be live, sound, straight and close-grained, cut square cornered, full length, not more than one-fourth inch scant in any dimension for rough timber or one-half inch for dressed timber, free from large, loose, or unsound knots, knots in groups, or other defects that will materially impair its strength for the purpose for which it is intended. Subject to inspection before loading.

Stringers. Shall show not less than 90 per cent heart on each side and edge, measured across the surface anywhere in the length of the piece. Shall be out of wind and free from shakes, splits, or pitch pockets over three-eighths inch wide or 5 inches long. Knots greater than 2 inches in diameter will not be permitted within one-fourth of the depth of the stringer from any corner nor upon the edge of any piece; knots shall in no case exceed 3 inches in diameter.

Standard Grade. Shall include yellow, red, and white Douglas fir and western hemlock.

General Requirements. All timbers shall be sound and cut square cornered, except that timbers 10 inches by 10 inches in size may have a 2-inch wane on one corner or its equivalent on two or more corners. Other sizes may have proportionate defects. Must be free from defects which will impair its utility for temporary work. Knots shall not exceed one-fourth the width of the surface of the piece in which they occur. Subject to inspection before loading.

Stringers. Shall be out of wind, free from shakes or splits extending over more than one-eighth of the length of the piece, or knots more than 4 inches in diameter. Knots greater than 3 inches in diameter will not be permitted on the edge of any stringer.

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* The American Society for Testing Materials has recently incorporated the density requirement of the Forest Service rules in its specifications for southern yellow pine structural timbers.

Grading Rules Proposed by Forest Service.

Specification E.

The following proposed rules were prepared at the Forest Products Laboratory to apply to southern yellow pine in structural sizes. The rules were prepared for southern yellow pine as it happened that more definite data were available on the relation between density and summerwood for the southern pines than in the case of other species used for structural purposes. The rules are applicable to all structural timber, except the requirements for quality, number of rings, and proportion of summerwood. Such requirements may vary with different species. The rules proposed by the Forest Service are based on observations made during various tests and on a study of the strength data with special reference to physical characteristics. The purpose of the rules is to classify timbers according to their mechanical properties.

PROPOSED RULES FOR GRADING STRUCTURAL TIMBERS OF SOUTHERN YELLOW PINE.

Grade I.

Requirements for quality, number of rings and proportion of summerwood:

1. Shall contain only sound wood.

2. Wood shall have a density indicated by the following characteristics:

Shall show on the cross section an average of not less than one-third summerwood, measured over the third, fourth, and fifth inches on a radial line from the pith. Timber with an average of less than six annual growth rings per inch shall show an average of not less than one-half summerwood. Contrast in color between summerwood and springwood shall be sharp.

In cases where timbers do not contain the pith, and it is impossible to locate it with any degree of accuracy by curvature of the rings, the same inspection shall be made over three inches of an approximately radial line beginning at the edge nearest the pith.

Restrictions on Knots in Beams:

3. Shall not have in Volume 1* sound knots with a diameter greater than 1½ inches or one-fourth the width of the face on which they appear,—or knots over one-half inch in diameter which are insecurely attached to the surrounding wood. Shall not have in Volume 2 sound knots with a diameter greater than 3 inches or one-half the width of the face on which they appear,—or insecurely attached knots with a diameter greater than 1½ or one-fourth the width of the face on which they appear.

The aggregate diameter of all knots within the center half of the length of any face shall not exceed the width of that face.

Note.—The diameter of a knot on the narrow or horizontal face of a beam is to be taken as its projection on a line perpendicular to the edge of the timber. On the wide or vertical face the smallest dimension of a knot is to be taken as its diameter. See Figure 22.

Restrictions on Knots in Columns:

4. Shall not have sound knots whose diameters are greater than 4 inches or one-third the least width of the column, or knots over one-half inch in diameter which are insecurely attached to the surrounding wood.

Note.—The diameter of a knot is to be taken as its projection on a line perpendicular to the edge of the column. See Figure 22.

Restrictions on Shakes and Checks in Beams:

5. Ring-shakes shall not occupy, at either end of a timber, more than one-fourth the width for green material, nor more than one-third the width for seasoned material.

Any combination of checks and shakes which would reduce the strength to a greater extent than the allowable ring-shakes will not be permitted. Shakes shall not show on the faces of either green or seasoned timber.

Note.—The importance of shakes and checks in the middle half of the height of a beam is dependent upon the magnitude of the horizontal shearing stress. The specification given is for beams in which the allowable shearing stress is developed. In beams whose length is more than 15 times their height the allowable

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* See Figure 22.

shearing stress is not usually developed and as the ratio of length to height increases, the importance of these shakes and checks decreases proportionately. Ring-shakes, showing on an end, should be considered as extending to the center. Checks and radial or star-shakes are not usually continuous in the direction of the length of the timber but should not be allowed if, in the judgment of the inspector, their weakening effect is greater than that of the allowable ring-shakes.

Allowable conditions for both green and seasoned material are given because checks are formed and shakes further developed during seasoning. The maximum amount of shakes and checks should not be allowed unless it is practically certain from the appearance of the timber, or from a knowledge of the seasoning conditions or length of time elapsed since cutting, that it is thoroughly seasoned. For purposes of this rule it will be considered that at least 1½ months per inch of thickness is required for material to become thoroughly seasoned.

Restrictions on Cross-Grain in Beams:

6. Shall not have diagonal grain with slope greater than one in twenty within the middle half of the length of the beam.

Grade II.

Grade II includes material rejected from Grade I on account of either (A) having less density than required for Grade I or (B) having more serious defects than allowed in Grade I.

(A) Material rejected from Grade I because of deficient density will be accepted in Grade II, provided it conforms to all the requirements of Grade I, except that in paragraph 2 the the requirements for *one-third* summerwood in material having six rings and over per inch, shall be changed to *one-fourth*; and that the requirements for *one-half* summerwood in material having less than six rings per inch, shall be changed to *one-third.*

(B) Material rejected from Grade I for having excess of serious defects will be accepted in Grade II, provided it conforms to paragraph 2 of Grade I, and has defects limited as follows:

Restrictions on Knots in Beams:

7. Shall not have in Volume 1 sound knots with a diameter greater than three inches or one-half the width of the face

on which they appear; or knots which are insecurely attached to the surrounding wood, whose diameter exceeds 1½ inches or one-half the width of the face on which they appear.

Shall not have in Volume 2 knots which are insecurely attached whose diameter exceeds 3 inches or one-half the width of the face on which they appear.

The aggregate diameter of all knots within the center half of the length of any face shall not exceed two times the width of that face.

Note.—For method of measuring diameters of knots in beams see note in Rule 3.

Restrictions on Knots in Columns:

8. Sound knots whose diameters are greater than 6 inches or one-half the least width of the column, or knots which are insecurely attached to the surrounding wood and whose diameters are greater than 3 inches or one-fourth the least width of the column shall not be permitted.

Note.—For method of measuring diameters of knots in columns see note in Rule 4.

Restrictions on Shakes and Checks in Beams.

9. Ring-shakes shall not occupy, at either end of a timber, more than one-third the width for green material, nor more than one-half the width for seasoned material.

Any combination of checks and shakes which would reduce the strength to a greater extent than the allowable ring-shakes shall not be permitted. (See note in Rule 5.)

Discussion of Grading Rules.

By comparing the few specifications given, it will be seen that there is a wide variation in the rules now in use. Some of them are very rigid and would reject a large amount of first-class material. Others are so loosely drawn that the selection of material is left largely to the judgment of the inspector. It will be seen that many of the points contained in the proposed Forest Service rules are found in other rules. The main point in which the Forest Service rule differs from others is in the emphasis which is laid on the position of defects and on the density of the material and the visual method of estimating

whether or not material under inspection is dense enough to be classed as Grade I or Grade II material or to be culled. In some cases where material has been graded by commercial rules it has been found*, after test, that the average strength of such material has been higher in the intermediate grade than in the highest grade, which, of course, indicates that the method of classification was unsatisfactory from the standpoint of strength.

STRENGTH TESTS MADE BY THE UNITED STATES FOREST SERVICE.

The methods of test used by the Forest Service have been described in a number of papers† and bulletins† and will not be given here in detail. Figures 23 to 30, inclusive, show the methods used in the various kinds of tests.

Table 3 of the Appendix presents average strength values for both full size market material and small, clear, straight-grained pieces cut from them. All the tests were made on green material. The average values for the large beams were in each case derived from structural material of both high and intermediate quality.

Relations Indicated by Tests.

1. The density or dry weight of wood is a measure of its strength.

2. Each annual growth ring is made up of a comparatively heavy band of summerwood and a lighter band of springwood. The greater the proportion of summerwood, the greater the weight and strength of the timber.

3. The number, character, and location of checks, shakes, and knots in timber have much to do with its strength.

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* Forest Service Bulletin 115 ''Mechanical Properties of Western Hemlock'' by O. P. M. Goss.

† Forest Service Bulletin 108 ''Tests of Structural Timbers''.

Forest Service Circular 38 ''Instructions to Engineers of Timber Tests'', Revised.

Paper presented at the Sixth Congress of International Association for Testing Materials ''Forest Service Investigation of American Woods with Special Reference to Investigations of Mechanical Properties'' by McGarvey Cline.

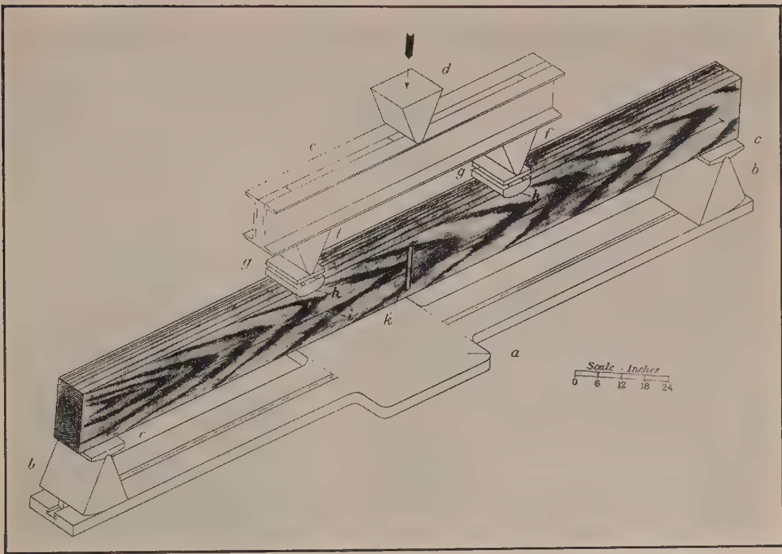

Fig. 23. Bending Test. 8″ x 16″ x 16′ Bridge Stringer.

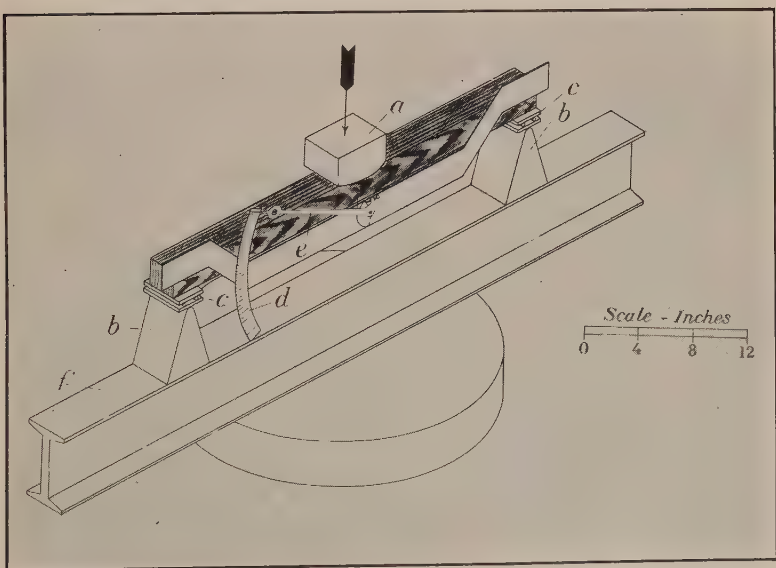

Fig. 24. Bending Test. Specimen 2″ x 2″ x 30″.

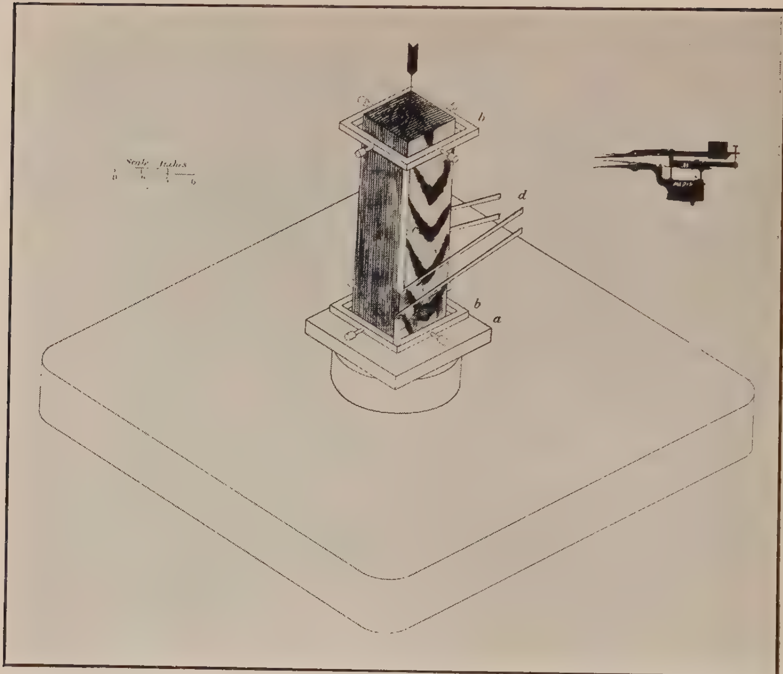

Fig. 25. Compression Parallel to Grain. Specimen 2″ x 2″ x 8″.

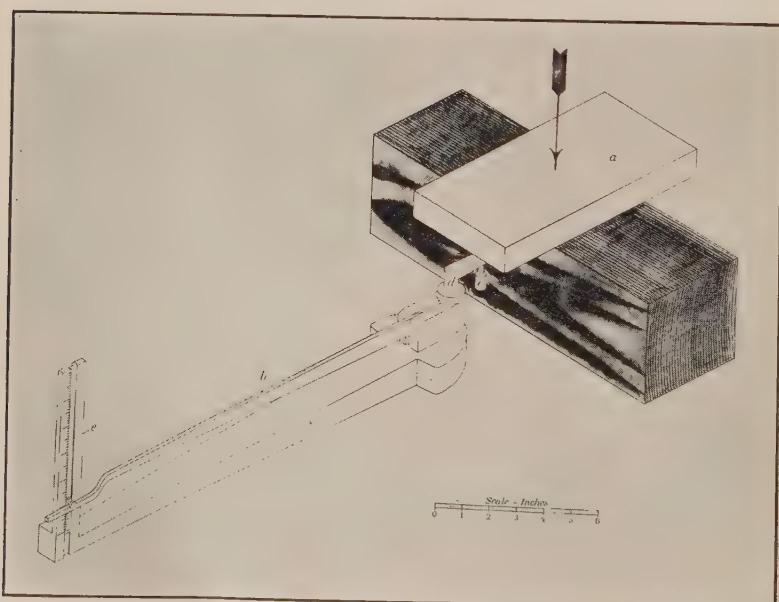

Fig. 26. Compression Perpendicular to Grain. Specimen 2″ x 2″ x 6″.

4. Checks and shakes in beams reduce the area which resists horizontal shear. Such defects are most harmful when they are in the center half of the height of the beam, as they

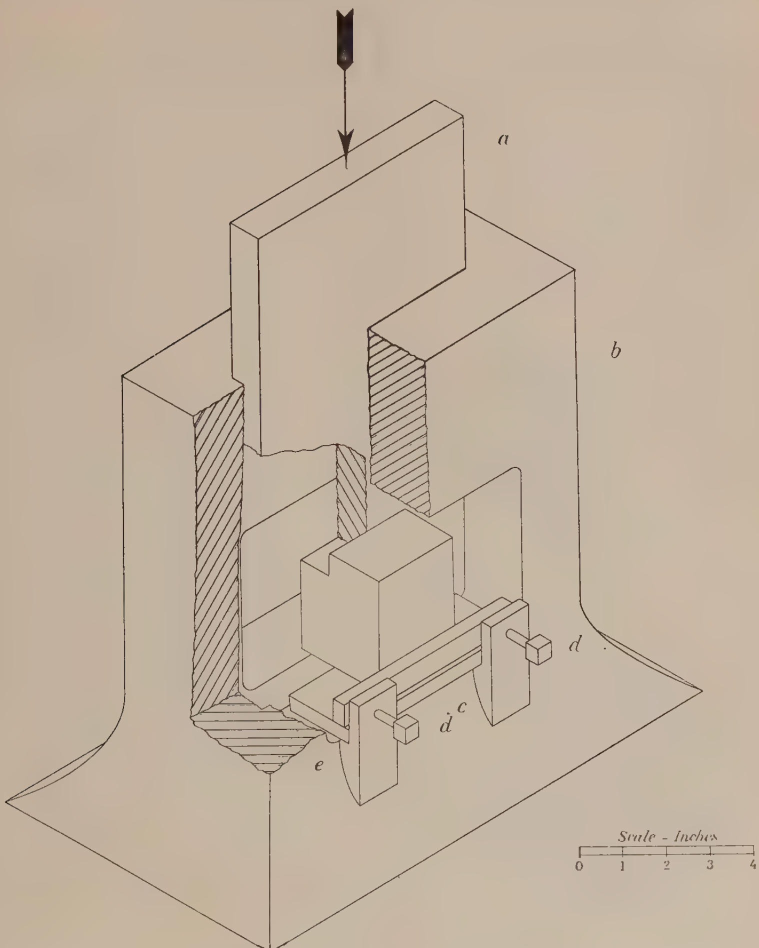

Fig. 27. Shearing Test. Specimen cut with projecting shoulder.

are then comparatively near the neutral plane, where their effect would be greatest.

5. The weakening effect of knots depends upon their position, soundness, tightness, and the amount they distort the

grain of the wood from a straight line. A comparatively small knot near the lower edge may be more harmful than a large knot located elsewhere.

6. No differences in mechanical properties due to a change from sap to heart have been found. As a general rule, in species which show a variation in the mechanical properties

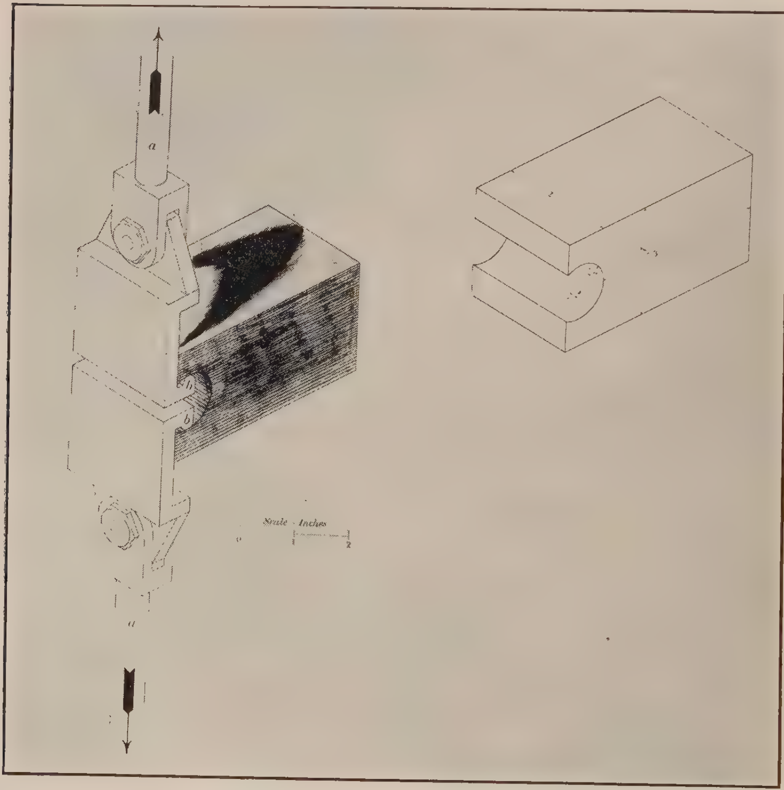

Fig. 28. Cleavability Test.

with position in cross sections, there is a certain age when the best wood is produced. In such species the age and thrift of the tree determine whether heart or sap is the best. For example, in a young, thrifty hickory, the sapwood will usually be the best, while in a large, over-mature tree of the same species, the heartwood will be the best.

7. Exceedingly rapid or slow growth in conifers has usually been found to be attended by lack of density and inferior mechanical properties.

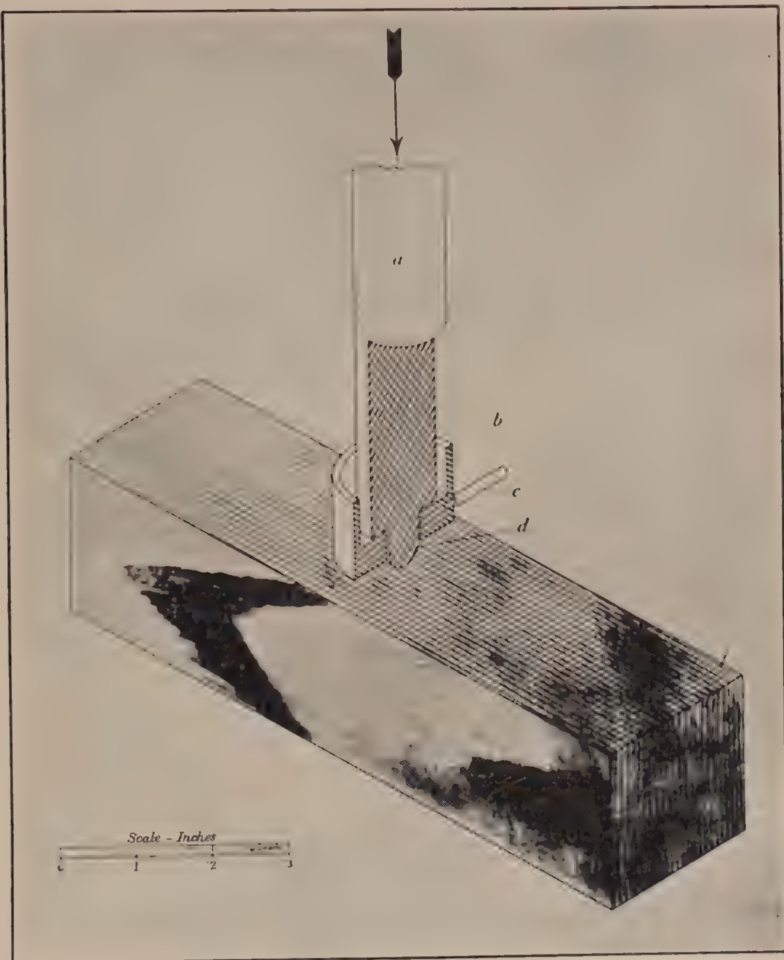

Fig. 29. Hardness Test. Specimen 2″ x 2″ x 6″.

8. The effect of location of growth on the nature of the timber is very complex. Variations attributed to difference in locality of growth are frequently exaggerated. These varia-

tions are generally apparent in the difference in density of the wood.

9. Trees growing close together and apparently under the same conditions occasionally show a difference in their mechanical properties that can not be entirely accounted for by the difference in density. Whether this difference is due to the ancestry of the tree or some other cause, such as soil conditions, is not yet known.

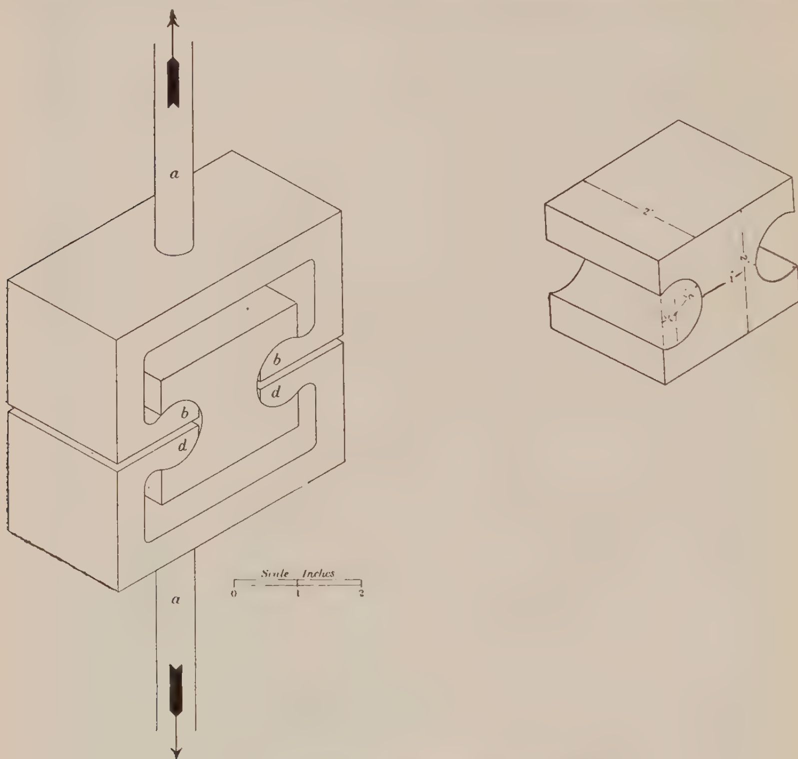

Fig. 30. Tension at Right Angles to Grain.

10. In tests on structural timber in commercial sizes, it has been found that large timbers in a green condition and containing no especially harmful defects (these might be called Grade I or select structural timbers) gave stresses about three-fourths as great as the small, clear pieces cut from them.

11. The strength of small, clear pieces is greatly increased by seasoning. In large timbers, the increased strength attending a loss of moisture is mostly offset by checks and other defects developed during the seasoning process, and therefore, under most conditions it is not considered advisable to anticipate any added strength due to seasoning.

TABLE 1.

Lumber—Number of Active Mills Reporting and Quantity of Lumber Sawed.

By States: 1913, 1912, 1911.

State	Number of Active Mills Reporting			Lumber Sawed Quantity (M feet b. m.)		
	1913	1912	1911	1913	1912	1911
United States	21,394	29,648	28,107	38,387,009	39,158,414	37,003,207
Washington	469	788	777	4,592,053	4,099,775	4,064,754
Louisiana	408	460	502	4,161,560	3,876,211	3,566,456
Mississippi	679	952	908	2,610,581	2,381,898	2,041,615
Oregon	406	480	522	2,098,467	1,916,160	1,803,698
Texas	341	450	430	2,081,471	1,902,201	1,681,080
North Carolina	1,206	2,418	2,071	1,957,258	2,193,308	1,798,724
Arkansas	808	1,145	1,127	1,911,647	1,821,811	1,777,303
Alabama	816	1,249	1,112	1,523,936	1,378,151	1,226,212
Wisconsin	612	792	771	1,493,353	1,498,876	1,761,986
Virginia	1,574	2,126	2,005	1,273,953	1,569,997	1,359,790
West Virginia	678	961	994	1,249,559	1,318,732	1,387,786
Michigan	532	792	796	1,222,983	1,488,827	1,466,754
California	141	229	222	1,183,380	1,203,059	1,207,561
Minnesota	354	484	467	1,149,704	1,436,726	1,485,015
Florida	203	397	295	1,055,047	1,067,525	983,824
Tennessee	1,155	1,567	1,536	872,311	932,572	914,579
Georgia	688	1,117	952	844,284	941,291	801,611
Maine	686	826	817	834,673	882,128	828,417
Pennsylvania	1,159	1,724	1,636	781,547	992,180	1,048,606
South Carolina	497	750	541	752,184	816,930	584,872

Idaho	161	202	209	652,616	713,575	765,670
Kentucky	1,061	1,386	1,452	541,531	641,296	632,415
New York	1,917	1,487	1,520	457,720	502,351	526,283
Missouri	844	1,210	1,112	416,608	422,470	418,586
Ohio	826	1,156	1,009	414,943	499,834	427,161
Montana	109	118	126	357,974	272,174	228,416
Indiana	695	978	915	332,993	401,017	360,613
New Hampshire	305	441	397	309,424	479,499	388,619
Massachusetts	312	420	430	224,580	259,329	273,317
Vermont	363	507	498	194,647	235,983	239,254
Maryland	258	404	389	140,469	174,320	144,078
Oklahoma	113	200	174	140,284	168,806	143,869
Illinois	275	463	394	102,902	122,528	96,651
Connecticut	177	215	218	93,730	109,251	124,661
Arizona	14	12	12	77,363	76,287	73,139
Colorado	89	135	156	74,602	88,451	95,908
New Mexico	28	41	47	65,818	82,650	83,728
New Jersey	94	146	136	27,248	34,810	28,639
Iowa	162	157	160	21,676	46,593	59,974
South Dakota	15	28	28	19,103	20,986	13,046
Delaware	44	86	82	18,039	28,285	23,853
Rhode Island	15	22	20	14,984	14,421	9,016
Wyoming	57	56	75	12,940	13,560	33,309
Utah	39	59	62	5,403	9,055	10,573
All other States*	9	12	5	19,461	22,525	11,786

* Includes Kansas, Nebraska, and Nevada.

TABLE 1.—Continued.

Lumber—Quantity of Lumber Sawed.

By Kinds of Wood: 1913,† 1912,* and 1911.*

Kind of wood	Quantity (M feet b. m.) 1913	1912	1911
Total	38,387,009	39,158,414	37,003,207
Yellow pine	14,839,363	14,737,052	12,896,706
Douglas fir	5,556,096	5,175,123	5,054,243
Oak	3,211,718	3,318,952	3,098,444
White pine	2,568,636	3,138,227	3,230,584
Hemlock	2,319,982	2,426,554	2,555,308
Western pine	1,258,528	1,219,444	1,330,700
Cypress	1,097,247	997,227	981,527
Spruce	1,046,816	1,238,600	1,261,728
Maple	901,487	1,020,864	951,667
Red gum	772,514	694,260	582,967
Tulip poplar	620,176	623,289	659,475
Redwood	510,271	496,796	489,768
Chestnut	505,802	554,230	529,022
Larch	395,273	407,064	368,216
Birch	378,739	388,272	432,571
Beech	365,501	435,250	403,881
Cedar	358,444	329,000	374,925
Basswood	257,102	296,717	304,621
Elm	214,532	262,141	236,108
Cottonwood	208,938	227,477	198,629
Ash	207,816	234,548	214,398
Hickory	162,980	278,757	240,217
Sugar pine	149,926	132,416	117,987
Tupelo	120,420	122,545	98,142
Balsam fir	93,752	84,261	83,375
White fir	88,109	122,613	124,307
Walnut	40,565	43,083	38,293
Sycamore	30,804	49,468	42,836
Lodgepole pine	20,106	22,039	33,014
All other kinds	85,366	82,145	69,548

† Compiled by the Bureau of Crop Estimates and the Office of Industrial Investigations of the Forest Service.

* Compiled by the Bureau of the Census and the Office of Industrial Investigations of the Forest Service.

TABLE 2.

Wood Used Annually by Principal Wood-using Industries in Per Cent of Total* Annual Consumption.†

(Average Annual Consumption, 52,000,000,000 Feet B. M.)

	Industry	Per cent of Annual Consumption
1.	Lumber and structural timber	27.6
2.	Planing mill products, sash, doors, and general mill work	25.6
3.	Boxes and crates	8.7
4.	Ties (Census 1911)	8.6
5.	Mine timbers (Census 1905)	4.6
6.	Pulp (Census 1911)	4.1
7.	Car construction	2.4
8.	Shingles (Census 1912)	2.3
9.	Furniture	1.8
10.	Vehicles	1.4
11.	Slack cooperage (Census 1911)	1.2
12.	Distillation (Census 1911)	1.2
13.	Lath (Census 1912)	1.0
14.	Tight cooperage (Census 1911)	.9
15.	Veneers (Census 1911)	.9
16.	Woodenware and novelties	.8
17.	Agricultural implements	.6
18.	Chairs	.6
19.	Handles	.5
20.	Musical instruments	.5
21.	Tanks and silos	.4
22.	Poles (Census 1911)	.4
23.	Ship and boat building	.4
24.	Fixtures	.4
25.	Caskets and coffins	.3
26.	Refrigerators and kitchen cabinets	.3
27.	Excelsior	.2
28.	Miscellaneous secondary industries (39) and extract wood	2.3
	Total	100.0

* Exclusive of fuel, fence posts, and rails.

† Prepared by the Office of Industrial Investigations of the Forest Service.

TABLE 3.

Average Strength Values, Green Structural Timbers With Ordinary Defects and Small Specimens of Green Material Without Defects.

Species	Weight per cubic foot, oven dry Lbs.	Rings per inch	Bending: Fiber stress at elastic limit per square inch Lbs.	Bending: Modulus of rupture per square inch Lbs.	Bending: Modulus of elasticity per square inch 1000 lbs.	Compression parallel to grain — Crushing strength at maximum load per square inch Lbs.	Compression perpendicular to grain — Crushing strength at elastic limit per square inch Lbs.	Shear — Shearing strength per square inch Lbs.
Cypress:								
Structural sizes	......	......	....	....	....	....	......	......
Small specimens	32.2	25.0	4430	7110	1378	3960	548	818
Douglas fir:								
Structural sizes	28.0	11.0	3968	5983	1517	3495	570	......
Small specimens	......	......	5227	8280	1597	4030	......	765
Eastern hemlock:								
Structural sizes	......	......	....	....	....	....	......	......
Small specimens	28.0	20.5	4155	6685	1124	3270	497	877
Loblolly pine:								
Structural sizes	31.0	5.9	3040	5084	1387	2940	500	......
Small specimens	......	......	4100	7870	1440	3240	......	630
Longleaf pine:								
Structural sizes	34.1	16.7	4015	6191	1600	....	495	......
Small specimens	35.1	16.5	5270	8330	1530	4285	570	1048

Norway pine:								
Structural sizes	25.0	13.7	2492	3864	1133	2555	......	......
Small specimens	......	......	2808	5173	960	2504	......	589
Red spruce:								
Structural sizes	25.5	21.9	2394	3566	1180	....	......	......
Small specimens	......	21.3	3627	5900	1157	2750	310	758
Redwood:								
Structural sizes	22.0	18.8	3760	4472	1042	3882	434	......
Small specimens	......	......	4750	6980	1061	3980	569	742
Shortleaf pine:								
Structural sizes	30.0	12.1	3237	5548	1473	3435	351	......
Small specimens	......	......	4350	7710	1395	3570	400	704
Tamarack:								
Structural sizes	30.0	14.0	2813	4556	1220	3230	......	......
Small specimens	......	......	3875	6820	1141	3190	......	668
Western hemlock:								
Structural sizes	27.0	15.6	3516	5296	1445	3355	434	......
Small specimens	......	......	4406	7294	1428	3392	......	630
Western larch:								
Structural sizes	28.0	24.3	3324	4948	1301	3510	456	......
Small specimens	......	......	4274	7251	1310	3696	......	700
Western yellow pine:								
Structural sizes	30.8	15.6	2769	4560	1243	2830	299	......
Small specimens	25.6	14.9	3156	5831	1178	2896	......	......
White spruce:								
Structural sizes	21.3	9.3	2239	3288	1081	....	......	......
Small specimens	......	10.2	3090	5185	998	2370	270	651

TABLE 4.

Nomenclature of Southern Pines.

Botanical and Accepted Common Names.

Pinus palustris	Pinus echinata	Pinus taeda
Longleaf pine	Shortleaf pine	Loblolly pine

Local, Market, or Lumbermen's Names.

Longleaf pine	Shortleaf pine	Loblolly pine
Southern yellow pine	Yellow pine (N. C., Va.)	Slash pine (Va., N. C.)
Southern hard pine	Shortleaved yellow pine	Cornstalk pine (Va.)
Southern heart pine	Shortleaved pine	Oldfield pine (Gulf region)
Pitch pine (Atlantic)	Virginia yellow pine	Rosemary pine (N. C., Va.)
Longleaved yellow pine (Atlantic)	Bull pine (Va.)	Shortleaved pine (Va., N. C., S. C.)
Longstraw pine (Atlantic)	North Carolina yellow pine	Bull pine (Texas, Gulf region)
North Carolina pitch pine	North Carolina pine	Virginia pine
Georgia yellow pine	Carolina pine	North Carolina pine
Georgia pine	Slash pine (N. C., Va.)	Sap pine (Va., N. C.)
Georgia heart pine	Oldfield pine (Ala., Miss.)	Yellow pine (No. Ala., N. C.)
Georgia longleaved pine	Spruce pine	Swamp pine (Va., N. C.)
Georgia pitch pine	Rosemary pine (N. C.)	Spruce pine (Va.)
Florida yellow pine		
Florida pine		
Florida longleaved pine		
Texas yellow pine		
Texas longleaved pine		

TABLE 5.

Botanical Characteristics of the Three Principal Pines of the Southern States.

	Longleaf pine	Shortleaf pine	Loblolly pine
Leaves	Three in a bundle, 9 to 12 (exceptionally 14 to 18) inches long.	Two and three in a bundle, 1⅜ to 4 inches long; commonly 2⅓ to 5 inches long.	Three in a bundle, 5 to 9 inches long.
Cones (open)	6 to 10 inches long; 4½ to 5 inches in diameter.	1¼ to 2½ inches long; 1⅛ to 1¾ inches in diameter.	2½ to 4⅕ inches long; 1¾ to 3 inches in diameter.
Cone scales	⅞ to 1 inch broad; tips much wrinkled; light chestnut-brown, gray with age.	5/16 to ⅜ (exceptionally ½) inch broad; tips light yellow-brown.	⅜ to ¾ inch broad; tips smooth; dull yellow-brown, or light reddish-brown.
Cone prickles	Very short; delicate; incurved.	Exceedingly short (1/10 inch); delicate; straight.	Short; stout at base.

TABLE 6.

Physical Characteristics of the Wood of the Principal Southern Pines.*

	Longleaf pine	Shortleaf pine	Loblolly pine
Diameter of pith of trunk	Average 0.18 inch. Range 0.12-0.26 inch.	Average 0.05 inch. Range 0.02-0.10 inch.	Average 0.11 inch. Range 0.02-0.20 inch.
Annual rings	Generally narrow beyond the first 2 or 3 inches from center; 12 to 30 rings per inch. Large proportion of summerwood.	Generally wide near center and narrow beyond the first 5 to 7 inches; 7 to 15 rings per inch. Small proportion of summerwood.	Variable; generally wide near center; 3 to 12 rings per inch. Moderate proportion of summerwood.
Sapwood	Narrow, rarely over 2 to 3 inches wide.	Generally wide; rarely less than 2½ inches and up to 6 inches wide.	Very wide; rarely less than 3 inches up to 7 inches wide.
Bark	Generally thin, 1/16 to ¾ inch.	Moderate thickness; ⅛ to 1 inch.	Generally thick; ¼ to 2 inches.
Resin	Very abundant; parts often turning to ''lightwood''.	Moderately abundant; pitchy near stumps and knots.	Between longleaf and shortleaf.
Average weight per cubic foot	Oven-dry, 37 lbs. Air-dry, 41 lbs.	Oven-dry, 30 lbs. Air-dry, 34 lbs.	Oven-dry, 31 lbs. Air-dry, 35 lbs.

* Condensed from a Forest Service report ''How to Distinguish the Wood of the Southern Pines'', by Arthur Koehler.

DISCUSSION

Mr. Ira H. Woolson,* Mem. Am. Soc. M. E. (by letter), expressed the belief that the authors' plan of indicating the source and available supply of our various varieties of ordinary structural timbers, and the brief description of the characteristics of such timbers will be very interesting and serviceable to architects, engineers, and builders. Mr. Woolson.

The Grading Rules for Structural Timbers based upon soundness and density will undoubtedly eventually remove much of the misunderstanding and confusion which have arisen in the past due to the practice of attempting to grade timbers on a basis of botanical species. This has been particularly true in regard to southern yellow pine, where the friction between seller and purchaser of this timber has been more or less constant owing to a misinterpretation of terms.

The National Board of Fire Underwriters, he states, are thorough believers in this method of classification, and through the courtesy of the United States Forest Service they were permitted to give an advance publication of these Grading Rules for Yellow Pine in the Revised Model Building Code recently issued by the National Board of Fire Underwriters, and it was their pleasure to cooperate in a small way in the framing of these rules. In prefacing this publication they said:

> "It is generally admitted that it is practically impossible to always recognize the different species of yellow pine after being sawed into timber. In the past all yellow pine has usually been grouped into two classes, longleaf and shortleaf, the latter, which is often called North Carolina pine, being made to cover not only the true shortleaf pine, but all other varieties, including loblolly.
>
> "This classification has proved unsatisfactory, because of the uncertainty of making even this simple separation upon a botanical basis. Timber from trees of one species often possesses the physical characteristics of another species. Consequently it has long been recognized that the suitability of yellow pine timber for structural purposes should be determined by the character of its grain and density, and not by its botanical name."

Minor differences still exist among timber experts in regard to the requirements of these rules, but they will doubtless be settled in the near future; and when such adjustments have been accomplished, purchasing contracts can be executed without confusion.

Mr. Woolson feels that the authors are to be congratulated upon having placed this publication before the Congress in such an instructive and attractive manner.

Mr. F. J. Hoxie,† Mem. Am. Soc. M. E. (by letter), states that it is universally agreed that the fundamental qualities in structural timber are Mr. Hoxie.

* Consulting Engineer, The National Board of Fire Underwriters, New York, N. Y.

† Engineer and Special Inspector, Assoc. Factory Mutual Insurance Cos., Boston, Mass.

Mr. Hoxie. strength and durability. The authors state that in order to properly select timber for any given use, knowledge of the influence of various physical characteristics on its strength and durability are necessary. They show that the relation between strength and density is governed by a well defined law. This study is in exactly the right direction and is an important step toward better timber specifications.

The several specifications quoted are probably among the best of those in common use and in them there is a lack of clear descriptions of the qualities required, owing to the fact that there is little definite knowledge of these qualities. This is particularly true in regard to durability. Soundness and heart wood are the only factors mentioned that can influence durability.

Living fungus in its early stages of development is practically impossible to detect by superficial examination in freshly cut lumber. Some of the more destructive of the fungi may go into a resting state, in which they are difficult to detect by ordinary means in older lumber; therefore, soundness, as the word is commonly understood, has more to do with strength than durability. Heart wood in the same, as well as in different varieties of wood, varies greatly in its powers of resistance to rot.

The fact that lumber cannot be expected to equal in uniformity such manufactured materials as steel or concrete is a reason for more careful selection, rather than an excuse for greater carelessness, both from the standpoint of the buyer and that of the seller. The saw-mill that puts all of its lumber into one grade will ultimately receive for it the price of the poorest rather than that of the average. The engineer who uses such lumber must employ a factor of safety, both in strength and durability, sufficient to cover the poorest. He would be justified in paying a higher price for more uniformly selected material, but he must be able to apply reliable tests to assure himself that the careful selection which he pays for has been made. A specification assuring lumber of unquestionable reliability will not only increase its price but extend its use in important structures.

Timber is frequently used under conditions for which its natural durability in no way fits it. Such use invariably results in discouraging the use of all timber. At a slightly increased first cost, far less than the cost of replacement, inferior timber could be made serviceable by chemical treatment.

More study into the causes which influence the rotting of wood is needed in order that specifications both for timber and preservative treatments can be written with a reasonable degree of assurance that the desired results will be obtained. This will require profound study into the chemical, as well as the physical, qualities which underlie the strength and durability of the various woods. It will also require the most painstaking observation of the habits and requirements of the several important wood-destroying fungi. Some work has been done

already in this direction in both this country and Germany. The relation between temperature and velocity of growth of some of the timber-destroying fungi has been studied. The fungus-resisting power of tannins and the action of certain acids and bases as foods or poisons to fungi have been investigated. The field is so broad that what has been done thus far is but a start. Knowledge of the preventive effect on rot of tannins, resins and high temperatures gained by these investigations is useful, but will be more useful when the limits of their powers of prevention are more clearly understood. Mr. Hoxie.

The limiting humidity requirements of the several common wood-destroying fungi are of the utmost importance and are undoubtedly influenced by the water-resisting powers of certain woods. In buildings where low humidities are maintained, varieties of wood can be used with safety which would rapidly rot in other buildings where higher relative humidities prevail. The lack of definite knowledge of this condition is now causing many thousands of dollars worth of mill roofs to rot each year.

Chemical treatments are applied blindly, with little accurate knowledge of their suitability for different varieties of wood and for service under varying moisture and heat conditions, and with less knowledge of the peculiarities of tolerance or susceptibility of the several varieties of timber-destroying fungi.

Present methods of testing the durability of untreated timber and timber treated by different chemical processes are too indefinite to give information of the necessary definiteness for making specifications. In fungus pits the humidity, which is a most important factor in the growth of the fungi, is only roughly controlled. Glass-jar and Petri-dish cultures give information of value for comparison, but they do not show the reaction which may take place between the wood and the treating material over long periods of time with varying conditions of heat and moisture; also they fail to give information on the result of slow evaporation, attraction of moisture by hygroscopic materials and resistance to moisture by resinous substances,—all of which are important factors in timber preservation, whether natural or artificial.

In order to determine with a degree of certainty, necessary for specifications, the limiting conditions of moisture, power of resistance of various timbers and specific antiseptic power of timber-treating materials, a building of good size is required, with several rooms in which the humidity and temperature can be very carefully regulated and in which full-sized beams can be placed, both in their natural state and treated with chemicals, each room containing timber infected with only one variety of timber-destroying fungus.

Mr. Hoxie believes that some will say such work is of little value for practical specifications for structural timber. However he points to the fact that such purely scientific chemical, as well as physical, investigation has placed specifications for steel and concrete on a firm founda-

Mr. Hoxie. tion, and he can see no reason why they cannot do a similar service for wood, notwithstanding the fact that it is not manufactured, but grows without uniformity. Such work is being carried on grandly by the experts of the United States Forest Products Laboratory and should receive the hearty commendation of all interests.

Mr. Goss. **Mr. O. P. M. Goss,*** Assoc. M. Am. Soc. C. E. (by letter), feels that this paper gives some valuable information regarding the timber resources of the United States and shows that this Government controls sufficient timber areas to insure a permanent supply. The brief history given of the milling operations throughout the United States is interesting. It is probably little realized by the layman that Douglas fir, a single species, composes more than twenty-five per cent of the total standing timber supply in the United States, including both softwoods and hardwoods (see Fig. 2). Practically all of this stand of Douglas fir is in the states of Washington and Oregon.

Since Douglas fir is to play so important a part in the future timber supply of this country, it is important to know the structural properties of this species as compared to other structural timbers of the United States. Perhaps the best known species for comparison is longleaf pine. Figure 19 shows the average modulus of rupture for all of the southern pines combined, to be approximately 7970 pounds per square inch. The same figure for Douglas fir from Washington and Oregon is 8280 pounds per square inch (see Table 3). The average strength, therefore, of the commercial southern pines as a group, based on modulus of rupture, is 96.2 percent as great as that of Douglas fir. Comparing the strength of the clear fibre of the single species, longleaf pine, with that of Douglas fir, based on modulus of rupture (Table 3) Douglas fir is 99.4 percent as strong as the longleaf pine.

He believes the proposed grading rule has been worked up along the right lines. The part of the rule dealing with the location of knots is especially practicable. More emphasis has been placed on the density of the wood, however, than is justified. In the first place, it is not possible, as a rule, to estimate the summer wood closer than five to ten percent. Even if it were possible to measure it to the finest degree of exactness, it would still not be thoroughly dependable in forecasting the strength of structural timber. It is a true indication of strength when applied to small clear specimens but is not equally true when applied to structural sizes. United States Forest Service Bulletin 108, Fig. 15, page 39, small clear specimens of Douglas fir, shows that with an increase in dry weight of from nineteen to thirty-six pounds per cubic foot, there is an accompanying increase in modulus of rupture of from 5500 to 10,500 pounds per square inch. These figures indicate increases of 47.6 and 47.2 percent respectively for strength and weight.

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* Consulting Engineer, Assoc. of Creosoting Cos. of the Pacific Coast and West Coast Lumber Mfrs. Assoc., Seattle, Wash.

Mr. Goss.

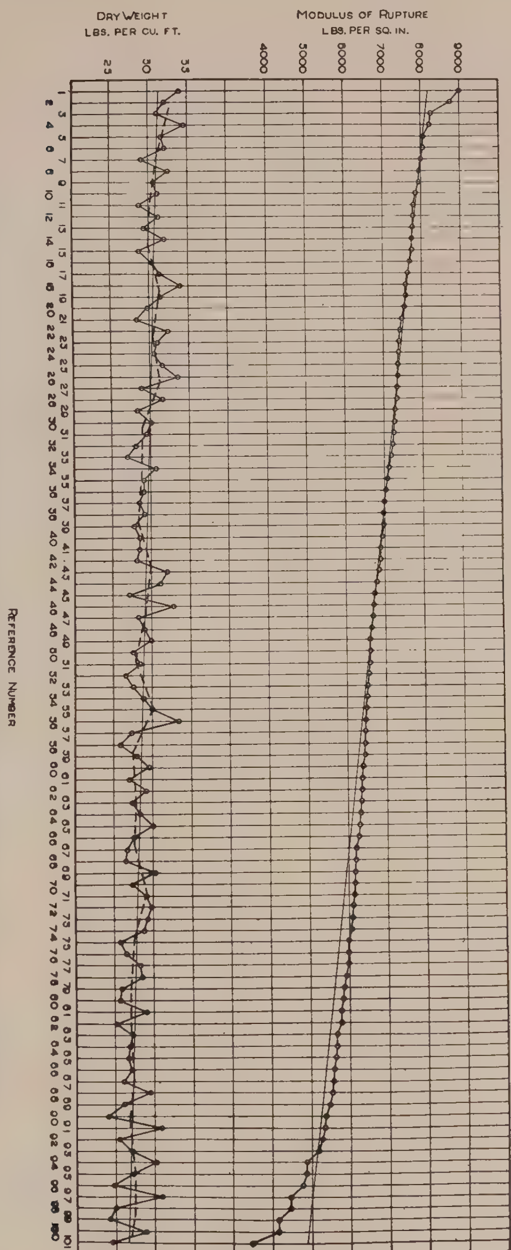

Fig. 31. Relation Between Modulus of Rupture and Dry Weight in Green Douglas Fir Bridge Stringers.

Mr. Goss.

This same bulletin shows in Figure 18, page 43, that when the modulus of rupture in structural sizes of longleaf pine varies on an average from 7600 lbs. to 4600 lbs. per square inch or 40 percent, no drop in weight occurs throughout this range in strength. As a matter of fact, the dry weight increases slightly in passing from the strongest to the weakest timbers.

Figures 31 and 32 herein have been prepared to show similar relations between strength and weight in structural sizes. The stringers used in these diagrams were chosen so as to include only those in which

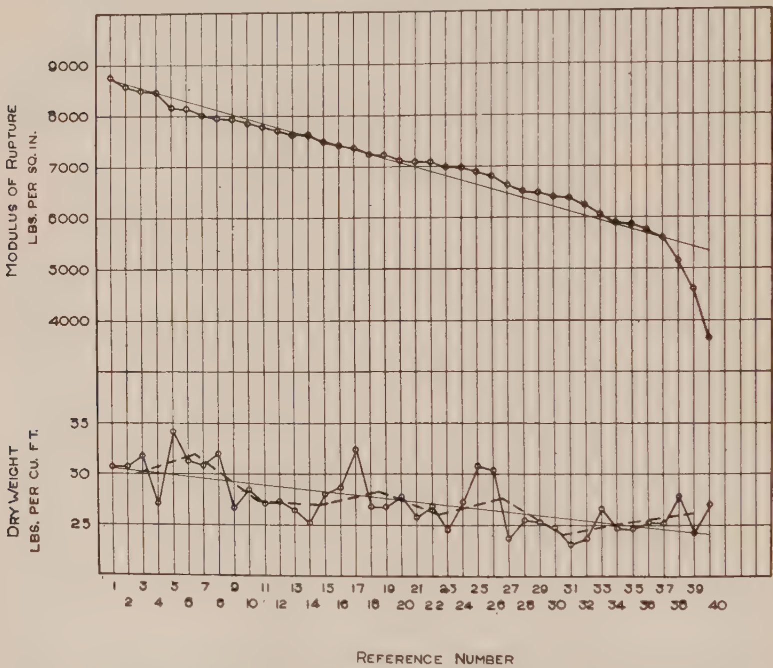

Fig. 32. Relation Between Modulus of Rupture and Dry Weight in Air-dry Douglas Fir Bridge Stringers.

the failure was not due to defects. If the strength-weight rule is true in any class of structural timbers, it would be true in the class here chosen. The weights used are true weights and are not estimated from percentage of summer wood.

These diagrams show results of tests of Douglas fir bridge stringers. The timbers have been arranged in order of their strength (modulus of rupture) in each diagram and the corresponding dry weights plotted in each case. Fig. 31 shows the results of tests of green 8 in. by 16 in. by 16 ft. timbers and Fig. 32 shows results for similar air-seasoned tim-

bers. It will be seen from Fig. 31 that with an average increase in strength of from 4880 to 8200 pounds per square inch, there is an average increase in dry weight of from 26.6 to 31.4 pounds per cubic foot. The increase in strength therefore is 40.5 percent while the increase in weight is but 15.3 percent. Mr. Goss.

Figure 32, air-seasoned timbers, shows that with an average increase in strength of from 5320 to 8710 pounds per square inch, there is an average increase in dry weight of from 24.1 to 30.5 pounds per cubic foot. With an increase in strength of 38.7 percent, there is an increase in weight of but 21.0 percent. From these figures it is seen that stringers do not follow the strength-weight law to anything like the same extent found in small, clear specimens.

In Figs. 31 and 32 the weight values often vary almost to extremes when no appreciable variation is found in the strength. Figure 31 shows no drop in weight over the last quarter of the curve where the drop in strength is very material. In Fig. 32 the last portion of the curve shows a marked increase in weight which is accompanied by a very decided drop in strength. In other words, the relation found between dry weight and strength in structural sizes is erratic and the dry weight cannot be depended upon, with any degree of certainty, to forecast the strength of structural timbers. For this reason it seems impractical to spend energy determining the amount of summer wood in structural timbers, other than to ascertain that from this standpoint the general quality of the stick is good.

Paper No. 96

TIMBER IN CANADA.

By

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Canada's present supply of commercial timber has been variously estimated as lying between five and seven hundred billion feet, board measure (500,000,000,000 to 700,000,000,-000), and covering an area of approximately 170,000,000 acres. This estimate of quantity and area refers only to timber of commercial value as saw timber. It does not include pulpwood, firewood, tie and pole material or small timber of any description, although this may have considerable commercial value.

Even pulpwood values are difficult to estimate as so much depends on accessibility to market. Firewood may be worth four dollars a cord in the settled parts of the country, and may have absolutely no value whatever in more remote districts. Ties may be worth forty cents at the railway but the cost of transporting them may exceed this value, and they then become valueless for the present at least.

A complete estimate of available forest products could not even be attempted with the information existing, and this estimate is therefore largely confined to commercial saw timber (including all material ten inches and over in diameter at the stump).

British Columbia contains a land area of approximately 226,186,240 acres (353,416 square miles), of which about twenty-one percent is covered with commercial saw timber. This area of about 50 million acres has been estimated to contain 300 billion feet, board measure.

The Coast type is made up largely of Douglas fir, hemlock, Sitka spruce, western red cedar, western tamarack, western white pine and others of less commercial importance, and contains the bulk of British Columbia's best saw timber (about 225 billion feet). The interior is divided into two distinct

types. The Dry Belt country is characterized by light precipitation and the tree growth is light in consequence. It consists largely of Douglas fir and western yellow pine. The Kootenay country has a high annual precipitation and is practically a modified repetition of the Coast type, characterized by the addition of such species as mountain fir and Engelmann spruce, and a lack of Sitka spruce. This type grades into the Southern Rocky Mountain type of mountain fir, Englemann spruce and lodgepole pine, which crosses the summit and clothes the eastern slope of the Rockies down to the prairie line.

British Columbia cut in 1913: 1,173,647,000 feet, board measure, of lumber in her mills. Over two thirds of this was Douglas fir, about seven percent was tamarack and seven percent red cedar, and of the remainder, five percent was spruce, five percent western yellow pine, three percent hemlock, two percent western white pine, and one percent each, mountain fir and jack pine. With the exception of unimportant quantities of cottonwood, maple and birch, no hardwoods are found in this province. Twelve kinds of wood were reported.

The lands in this province are under the administration of the Provincial Government, with the exception of the Railway Belt and the Peace River Block, which are under the administration of the Federal Government.

The reservation from sale or pre-emption of all lands carrying 8,000 board feet per acre west of the Cascade Mountains, or 5,000 board feet per acre east of this range, replaces the creation of formal reserves in that part of the province under provincial administration. This area is said to exceed 100,000,000 acres.

The Railway Belt consists of the land lying within twenty miles of the Canadian Pacific Railway's right-of-way across the Province. This belt which is forty miles wide and about five hundred miles long (containing about 12,800,000 acres in all), is said to contain 2,500,000 acres of saw timber or approximately thirty-six billion feet, board measure, of standing timber. In this district thirteen reserves have been set aside containing 2,420,838 acres in all. These are protected and administered by the Federal Government, as are also the forests in the Prairie Provinces, Alberta, Saskatchewan and Manitoba, while the

forests in all the Eastern Provinces are administered by the several Provincial Governments.

The Peace River Block of 3,500,000 acres has been estimated as 175,000 acres of saw timber or about one billion feet. This last is located in the northern Rocky Mountain region of the Province and the timber consists largely of spruce, mountain fir and lodgepole pine. So far none of the lands in the Block have been set aside as forest reserves.

The Province of Alberta has a total land area of 161,872,000 acres (252,925 square miles), of which 5,416,000 acres are said to contain saw timber to the extent of twenty-one billion board feet. Four reserves exist in Alberta at the present time, and these contain 16,711,776 acres. The largest of these, the Rocky Mountains Reserve, consists of all the non-agricultural land on the east slope of the Rocky Mountains. It covers an area of 14,474,856 acres, nine percent of which is saw timber land containing about eight billion feet, board measure. This province sawed in 1913, 44,662,000 feet of lumber of the following composition: spruce, 93.8 percent, jack pine, five percent, and small quantities of Douglas fir, tamarack, poplar, balsam fir and birch. With the exception of birch and poplar the forests of Alberta are entirely coniferous. The Douglas fir, Engelmann spruce, mountain fir and lodgepole pine, extend from British Columbia down the eastern slope and mix with the typically northern forest type.

In the saw mill returns these mountain species are mixed with those of the northern type of forest. Engelmann spruce is sold mixed with white spruce, mountain fir with balsam fir, and lodgepole pine with jack pine.

The northern forest type covers the country from the Rockies to the Atlantic and extends northward to the limit of tree growth. The type consists largely of white and black spruce, jack pine, balsam, and aspen poplar, with balsam fir and tamarack in smaller quantities. Toward the north this type of forest is of no commercial value except for purely local purposes and much of it is useless except for firewood and fencing. The trees become smaller and more gnarled and twisted until the place of the forest is taken by the treeless tundra or barren lands.

Further south this type improves and provides a great part of the consumption, at least of rough lumber, in the three prairie provinces. With the exception of the poplars and white birch this is a coniferous forest. The eastern hardwoods are entirely absent from this type. In northern Ontario and Quebec this type merges into the southern Laurentian type of mixed conifers and hardwoods.

In the northwest territories and the Yukon the forest at the present time has practically no commercial value. Forest cover exists but the trees are not found in commercial sizes or quantities. Wood is used locally for fuel, fencing and rough construction, but none is sawn and brought to the lumber markets. The country is sparsely settled; much of it has never been explored. Enormous areas have been burned repeatedly by forest fires and a considerable portion of the most northern part is composed of treeless barren lands or tundra. As a source of lumber supply for the future this district cannot be expected to supply more than the purely local demand.

Saskatchewan's land area is 155,764,080 acres (243,382 square miles) and the province's timber area covers 3,584,000 acres and contains about fourteen billion board feet. The province in 1913 cut 114,800,000 board feet of lumber. The lumber was made up of spruce (98.2 percent) almost entirely, with small unimportant quantities of tamarack, jack pine and poplar. The forest reserves in Saskatchewan cover an area of 1,152,889 acres and consist of eight different reserves, the largest of which (Porcupine No. 2) covers 361,440 acres.

Manitoba now contains a land area of 148,432,640 acres (231,926 square miles), of which 1,920,000 acres are saw timber land with a stand of some 6,850,000,000 feet of timber. The northern part of this province is covered with the same northern forest type found in Saskatchewan and eastern Alberta, but another type is found in the southeastern part. This is sometimes called the southern Laurentian type. The type proper is characterized by white and red pine, cedar and hemlock among the conifers and such eastern hardwoods as maple, yellow birch, elm, ash, oak, basswood, beech, etc. The white and red pine reach Manitoba as do also the cedar, black ash, white elm and basswood; the other trees of this type are usually confined to

Ontario and the provinces farther east. While none of these trees are commercially important in Manitoba, they indicate the transition stage between the northern coniferous forest and the northeastern hardwood type.

Green ash, Manitoba maple and burr oak are common in Manitoba and their range extends into Saskatchewan along the river bottoms, although they are not commercially important in the prairie provinces.

Manitoba in 1913 cut a total of 71,961,000 feet of lumber in her saw mills. Spruce formed about 90 percent of this quantity as in the other prairie provinces. Smaller quantities of poplar, tamarack, jack pine, birch and balsam fir are also produced. Five of the eastern species have also been reported from this province: these were cedar, oak, elm, white and red pine, although the quantities cut were never commercially important.

Five forest reserves have been set aside in this province containing an area of 2,629,440 acres. These reserves in the prairie provinces do not contain much of what would be classed as saw timber, but they contain promising young growth which is being protected from fire and managed on forestry principles and will in time produce saw timber. Many of these reserves are situated at stream sources to protect these and to insure steady stream-flow throughout the year. All of them contain firewood and posts which are of great value to settlers in the immediate neighborhood, especially in those parts of the country where tree growth is scarce.

The saw timber on all Dominion Forest Reserves is sold when it becomes mature. The timber is estimated, the sale advertised and the right to cut timber sold by tender.

Ontario is Canada's largest lumber producing province and probably contains more mature standing timber than any other province excepting, perhaps, British Columbia. The land area of Ontario has been estimated at 234,163,200 acres (365,880 square miles). The greater part of this area is covered with the northern forest type of spruce, jack pine, poplar, balsam fir and tamarack, and parts of the northwestern portion of the province are treeless or covered with timber of no commercial value. This northern type in Ontario covers at least 180 million acres. South of this in Ontario and, generally speaking, south

of the height of land between the St. Lawrence and Hudson Bay basins, we find the southern Laurentian type of forest which covers the southern rim of the Laurentian shield of rock formation. The rock is all igneous and has been worn by glacial action to a typical peneplain. The rock is seldom far from the surface and over large areas there is little or no soil cover at all. The type is characterized by the white and red pine, hemlock, cedar and the characteristic trees of the northern forest, together with such hardwoods as yellow birch, hard maple, ash, elm, basswood and some red oak and beech on the southern fringe of the type. The jack pine covers enormous areas of sandy plains and rocky country. White and red pine are generally found in deeper, moister soil, and hemlock, cedar and the hardwoods are confined to the moist rich bottom lands in this type. The southern Laurentian type covers some hundred million acres in Ontario and Quebec, and has been estimated to contain 200 billion feet of saw timber. In this area are situated the principal forest reserves of both these provinces. Ontario has a forest reserve area of 11,539,200 acres (18,030 square miles) and Quebec has 111,400,320 acres (174,062 square miles). Ontario is Canada's premier white pine province and the stand of this species has been estimated at about 40 billion feet for this province. The productive forest area in Ontario probably consists of from 70 to 90 million acres. The greater part of the newly added territory (the district of Patricia) is not covered with commercial timber, and is probably not capable of producing more than pulpwood.

South of the southern Laurentian type we find the northern fringe of the great central hardwood type of the United States. This type of forest covers the middle west and eastern states of the Union and extends across the boundary to Southern Quebec and Ontario.

The conifers, such as white and red pine, hemlock, cedar, etc., are still common, although the balsam fir, spruce and jack pine begin to disappear. The greater part of the forest is made up of hardwoods such as maple, birch, elm, oak, ash, beech, hickory and basswood, with smaller quantities of walnut, cherry, tulip, chestnut, butternut, black gum, sycamore and sassafrass. These last species are confined to the St. Lawrence Valley and

in some cases to a narrow strip of territory along the north shore of Lake Erie. The area covered by such species is small and unimportant in relation to the forest area of the province, being at the northern limit of their range. They grow largely on agricultural land and once removed will, in all probability, never be planted or reproduced in commercial quantities.

Ontario in 1913 cut 1,101,066,000 board feet of lumber, of which white pine formed about half. The cut was made up as follows: white pine 46.9 percent, hemlock 13 percent, red pine 12.0 percent, spruce 9.5 percent, maple (the most important hardwood) 5.6 percent, and twenty-two other kinds of wood, making a total of twenty-seven kinds.

Quebec with its recently added territory now contains a land area of 442,153,600 acres (690,865 square miles). Of this, about 367 million acres belong to the Northern Forest type of pure conifers, 50 million to the southern Laurentian type of conifers with mixed hardwoods and about five million acres to the hardwood type. The eastern counties of Quebec, south of the St. Lawrence, belong to another type which is characteristic of the Maritime Provinces, and is similar to the southern Laurentian. This type in Quebec covers about twenty million acres.

The Quebec government has reserved, as before stated, 111,400,320 acres (114,063 square miles) of forest land. The greater part of this lies in the northern portion of the Province, either in southern Laurentian or Northern Forest type and most of it is not heavily timbered.

Quebec in 1913 cut 630,346,000 feet of lumber. Spruce here forms 65.4 percent of the total, white pine only 11.4 percent and hemlock 6.1 per cent; birch comes fourth on the list with 5.4 percent, and is the most important hardwood. Generally speaking the rest of the lumber output is similar in composition to that of Ontario. Tulip, walnut, black gum, willow, ironwood and sycamore cut in Ontario are not reported from Quebec.

The provinces of New Brunswick and Nova Scotia and the eastern counties of Quebec, or in short that part of Canada lying south of the St. Lawrence River, is covered by a forest type often called the Acadian. This consists chiefly of "the hardwoods" birch, maple and beech, with smaller quantities of

basswood, ash, elm, oak and butternut. Those hardwoods which belong properly to the central hardwood forest, such as tulip, hickory, walnut, chestnut, sycamore, cherry and black gum are normally absent from this type. Red spruce is the most important conifer as compared to white pine in the southern Laurentian, and white spruce in the northern forest type. White and red pine are found in the Acadian type often in great abundance, but pure stands are scarce and most of the best material has been removed. The forest area might make up a total of 14 million acres, and is supposed to contain in round figures 100 billion feet of lumber. There are no forest reserves in the Maritime Provinces.

New Brunswick has a land area of 17,863,040 acres (27,911 square miles). The forest area has been estimated at 12 million acres, but this of course includes more than commercial saw timber land. The standing timber has been estimated at 22 billion feet of the following composition: spruce 60 percent, pine 10 percent, hemlock 5 percent, cedar 5 percent and hardwoods 20 percent. With the spruce in this estimate would be included balsam fir which is often sold mixed with spruce. New Brunswick in 1913 cut 399,247,000 feet of lumber of the following kinds: spruce 79.3 percent, white pine 7.8 percent, hemlock 5.5 percent, balsam fir 4.3 percent and birch (the most important hardwood) 1.4 percent. Ten other woods were cut in smaller quantities.

Nova Scotia's land area is 13,483,520 acres (21,068 square miles), and as a forest survey has been made of this province its forest conditions are better known than those in other parts of Canada.

The forest area has been estimated at 5,744,000 acres, and the coniferous saw timber at ten billion feet board measure. The hardwoods might provide five billion feet, making the total for the province about 15 billion feet. The standing timber (conifers) would have the following composition: red spruce five billion feet, hemlock three billion, white pine one billion, and the remainder, balsam fir, tamarack, red and jack pine, and white and black spruce. The hardwoods would be beech 40 percent, sugar maple 30 percent, yellow birch 20 percent, and white and wire birch, soft maple, red oak, white ash and black

ash, the remaining 10 percent. Nova Scotia cut about 274,722,000 board feet of lumber in 1913. Spruce formed 56.9 percent and hemlock 23.2 percent of this total. Seventeen kinds of wood in all have been reported from this province.

The forest area of Prince Edward Island is too small to be considered in a general estimate of this sort as the entire area of the province is only 1,397,760 acres (2,184 square miles). The annual production is 6,771,000 board feet, of which spruce forms a half and balsam fir a quarter. Fifteen kinds of wood in all were reported in 1912.

CANADIAN TREE SPECIES.

CONIFERS.

Pinus Strobus	White pine.
monticola	Western white pine.
flexilis	Limber pine.
albicaulis	White-barked pine.
resinosa	Red pine.
ponderosa	Western yellow pine.
Murrayana	Lodgepole pine.
rigida	Pitch pine.
Banksiana	Jack pine.
Larix laricina	Tamarack.
occidentalis	Western larch.
Lyallii	Alpine larch.
Picea Mariana	Black spruce.
rubra	Red spruce.
canadensis	White spruce.
Engelmannii	Engelmann spruce.
sitchensis	Sitka spruce.
Tsuga canadensis	Hemlock.
heterophylla	Western hemlock.
Mertensiana	Black Hemlock.
Pseudotsuga mucronata	Douglas fir.
Abies balsamea	Balsam fir.
lasiocarpa	Alpine fir.
grandis	Lowland fir.
amabilis	Amabilis fir.
Thuya occidentalis	Cedar.
plicata	Western cedar.
Chamaecyparis Nootkatensis	Yellow cypress.
Juniperus virginiana	Red juniper.
scopulorum	Rocky Mountain juniper.
communis	Juniper.
Taxus brevifolia	Western yew.

HARDWOODS.

Juglans nigra	Black walnut.
cinerea	Butternut.
Carya cordiformis	Bitternut hickory.
ovata	Shagbark hickory.
alba	Mockernut hickory.
glabra	Pignut hickory.
Populus tremuloides	Aspen.
grandidentata	Large-toothed aspen.
balsamifera	Balsam poplar.
acuminata	Lance-leafed cottonwood.
angustifolia	Narrow-leafed cottonwood.
deltoides	Cottonwood.
trichocarpa	Black cottonwood.
Salix sp.	Willow.
Ostrya virginiana	Ironwood.
Carpinus caroliniana	Blue beech.
Alnus incana	Speckled alder (shrub only).
oregona	Red alder.
sitchensis	Sitka alder.
tenuifolia	Mountain alder.
Betula populifolia	White birch.
alba var papyrifera	Paper birch.
occidentalis	Western birch.
lutea	Yellow birch.
lenta	Sweet birch.
Alaskana	Alaska birch.
fontinalis	Mountain birch.
Fagus grandifolia	Beech.
Castanea dentata	Chestnut.
Quercus alba	White oak.
Garryana	Garry oak.
stellata	Post oak.
macrocarpa	Burr oak.
prinus	Chestnut oak.
acuminata	Chinquapin oak.
prinoides	Dwarf chinquapin oak.
bicolor	Swamp white oak.
rubra	Red oak.
coccinea	Scarlet oak.
velutina	Black oak.
palustris	Pin oak.
Ulmus fulva	Red elm.
americana	White elm.
racemosa	Rock elm.
Celtis occidentalis	Hackberry.

Morus rubra	Red mulberry
Magnolia acuminata	Cucumber tree.
Liriodendron tulipifera	Tulip tree.
Asimina triloba	Papaw.
Sassafras variifolium	Sassafras.
Hamamelis virginiana	Witch hazel.
Platanus occidentalis	Sycamore.
Pyrus americana	Mountain ash.
Amelanchier canadensis	Service berry.
alnifolia	Saskatoon.
Crataegus sp.	Hawthorn.
Prunus nigra	Canada plum.
emarginata	Bitter cherry.
pennsylvanica	Bird cherry.
virginiana	Choke cherry.
demissa	Western choke cherry.
serotina	Black cherry.
Cercis canadensis	Red bud.
Gymnocladus dioica	Kentucky coffee tree
Rhus typhina	Staghorn sumach
Acer spicatum	Mountain maple.
pennsylvanicum	Striped maple.
macrophyllum	Broad-leafed maple.
circinatum	Vine maple.
Douglasii	Dwarf maple.
Acer saccharum	Sugar maple.
saccharinum	Silver maple.
rubrum	Red maple.
Negundo	Manitoba maple.
Tilia americana	Basswood.
Nyssa sylvatica	Black gum.
Fraxinus quadrangulata	Blue ash.
nigra	Black ash.
americana	White ash.
pennsylvanica	Red ash.
" var lanceolata	Green ash.

Paper No. 97

INDIAN TIMBERS USED IN ENGINEERING CONSTRUCTION.

By

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I. GENERAL.

Factors of Locality.

There are few countries which contain a larger and more varied flora than British India, and for this reason, every variety of climate is met with, from the intense cold and snows of the higher Himalayas to the damp, hot, tropical climate of the wet zone. Between these two extremes there exist many varying conditions, such as the moist zone just above the Ghats, which gradually merges into the dry zone conditions of the Deccan and Central India and from there into the deserts of Rajputana and Sind. The other factors governing the distribution of tree growth in India are, the extremes of altitude, which vary from sea level to over 14,000 feet; the many types of soil met with; and that the country extends from latitude 8° to latitude 37°.

Species.

Under the conditions referred to above, it is not surprising that the number of different species of trees found in India and Burma should amount to, roughly, 2500, and the species of woody shrubs to about an equal number. Of the former, only a relatively small number are suitable for engineering construction, though some of them yield some of the strongest and most durable timbers of the world.

II. SPECIES, SUPPLIES AND MARKET RATES.

Classification of Timber.

In a brief note it is not possible to describe more than a few timbers—and only those of greatest importance—but as a guide to the general resources of the country, in timber suitable for

engineering purposes, it may be stated that eight species come under the class, twenty-three are of considerable importance, and at least twenty-two other timbers are used for minor works, in default of better substitutes or for the sake of economy. The eight most important species are briefly described as below.

TEAK.

Teak, *Tectona grandis,* stands out preeminently as the first amongst the timbers of British India.

Distribution.

It occupies two distinct regions, one in the Peninsula and the other in the interior of Burma. In the western area, its northern limit is latitude 23° and its southern limit about latitude 10°; in Burma, it extends from latitude 25° southward to below Moulmein.

Growth.

In Burma and also on the West Coast of India, teak grows to a great size; trees of over 15′ girth are not infrequently found, though the economic exploitable girth is generally put at 7′ to 8′ in these localities. Elsewhere in India, except in the Chanda Division of the Central Provinces, the Surat Dangs and the Annavallies, where teak grows to 7′ and 8′ in girth, this species attains a girth of 3′ to 5′, with a comparatively short bole.

Physical and Mechanical Properties.

Teak timber is of a golden-brown colour when freshly cut, turning darker with age. Good Burma or Malabar teak is of fairly uniform colour, though occasionally streaked. Teak from the dry zone of the Peninsula of India is often beautifully marked with dark, wavy, irregular bands. It is moderately hard, easily worked, and extremely durable, which is probably due to the large quantity of tar oil it contains. It seasons slowly, but well, taking from 4 to 6 years to become thoroughly air dry. It shrinks during seasoning and does not settle down until the moisture in the wood is reduced to about 10%. Well-seasoned, air-dried teak weighs 45 lbs. per cubic foot. The average strength, according to tests recently carried out, is 7.66 tons per square inch for transverse strain applied tangentially and 6.91 tons per square inch when the pressure is applied radially to the annual rings; the amount of moisture in the timber at the time

of testing being 10%. Similar tests for shearing gave 0.736 tons per square inch, and for compression, 2.85 tons per square inch. A careful investigation recently carried out has shown that the timber from plantation and natural-grown Burma teak, obtained from the same locality, and of approximately equal age, differs in no way in point of strength.

Uses.

It is extensively used for building, in the form of beams, rafters, battens, posts and boards; for decks of ships and backing to armour plates of battleships; for boat-building, masts, spars and oars; railway carriages and waggons, cart-building and carriage-making in all its parts; railway sleepers—but for this purpose it is now too valuable; it is used in all departments of carpentry, for agricultural implements, coopers' work, turnery and many minor purposes.

Outturn.

The total outturn of teak in British India amounts to between 280,000 and 290,000 tons per annum, of which Burma supplies 225,000 tons and India the balance. The amount of teak exported from British India in 1913-14 amounted to 50,737 tons, of which a little over half found its way to England; the next most important countries being in order, Germany, Cape Colony, Japan and Ceylon.

Prices.

The price of teak varies considerably according to quality and the locality from which it is obtained. Thus, Burma and Malabar, or West Coast, teak sells at higher price than that obtained from the Deccan and the Central Provinces. An average price for unselected logs in the round in the Rangoon Depot may be put at * Rs. 100 to 115 per ton, the West Coast timber in the round at a depot on the seacoast fetches a shade over Rs. 100 per ton of 50 cubic feet. Central India and Deccan teak realise anywhere from Rs. 35 to Rs. 60 per ton. Best converted material in Calcutta fetches as much as Rs. 6 per cubic foot.

SÂL.

After teak, the next most valuable species available in India is Sâl, *Shorea robusta.*

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* Rupee = 1 shilling and 4 pence = 32 cents.

Distribution and Growth.

It occurs in India only, and not in Burma, occupying two distinct localities; one belt is situated at the foot of the Himalayas, starting from the Jumna River and running eastward through the United Provinces, Nepal into Bengal and Assam, and the other running through the Central Provinces, Bihar and Orissa, into a northern district of Madras. Sâl is found at its best in the east of the United Provinces, in Nepal and Bengal, where it grows to a height of 100 feet and more and attains a girth of 12 to 15 feet, while examples are cited of trees growing to 150 feet in height with a girth of 20 to 25 feet at three and a half feet from the ground; these, however, would be exceptionally large trees.

Physical and Mechanical Properties.

Sâl may be termed a coarse-grained timber when compared with teak. In colour, it is red-brown—pale when freshly cut and darkening with age. It is close-grained, hard, and weighs 50 lbs. per cubic foot. The fibre is arranged in alternate bands, one band being made up of several years of growth, and is so arranged that the fibre is more or less parallel to the axis of the stem but oblique to the next layer, hence the timber is difficult to split on the radial cut. The timber is extremely durable, many instances being recorded of its lasting in water, under ground and under cover of upwards of 100 years. As sleepers, it lasts untreated for from 16 to 18 years. It seasons very slowly, taking 6 to 8 years to do so in the log, during which time it develops many surface cracks. It is about equal to teak in strength to withstand transverse strain. The average of 24 tests on Sâl timber grown in different localities gave its transverse strength at 7.256 tons per square inch, the moisture in the timber at the time of testing being between 5% and 14%; while the shearing tests gave 0.844 tons per square inch and compression tests gave 3.55 tons per square inch, both figures being considerably above that for teak.

Uses.

Sâl timber is primarily used for construction, and next for railway sleepers. The annual number of Sâl broad- and metre-gauge sleepers used by the railway systems of India, based on an average of 3 to 11 years was 1,131,497. In construction, it is gen-

erally used as beams, rafters and planks. It is also used extensively as piles, for cart construction, agricultural implements, dugouts, helms, masts, oars, and in coopers' work for large vats. It is used for waggons, buffers, beams and brake-blocks in railway construction and for carts, wheel-barrows, etc., by the Ordnance Department.

Outturn.

The total annual outturn of Sâl timber is computed to be as follows:

From State Forests	6,337,736	c. ft. in 1910-11
22 Feudatory States of Orissa	966,764	c. ft. average of 3 years
Zemindari lands of the Central Provinces	50,000	c. ft. estimated
Mechpara Estates in Bengal	112,327	c. ft. average of 3 years
Private Sâl forests in Madras	50,000	c. ft. estimated
Total	8,120,551	cubic feet

Of the above amount, 32% goes to sleeper woods and 68% to constructional timber.

Prices.

Sâl from the northern forest, which is generally termed "Hill" or Nepal Sâl, fetches about Rs. 70 to Rs. 75 per ton of 50 cubic feet when converted into sleepers; its price in the round varies between Rs. 25 and Rs. 35 per ton, and as rafters of the larger size, 8 annas* to 12 annas per cubic foot.

DEODAR.

Distribution and Growth.

The most important conifer of India is the Deodar, *Cedrus deodara,* found in the Western Himalayas from Afghanistan eastwards to Kumaon, at from 4000 to 10,000 feet elevation, but most common at an elevation of from 6000 to 8000 feet. It grows to a large size, the extreme girth mentioned by writers is 44 feet at 2 feet and 36 feet at 6 feet from the ground, and 150 feet in height. Trees of such dimensions are now rare, the economic exploitable girth being fixed at about 8 feet at breast height.

Physical and Mechanical Properties.

The timber of Deodar is light yellow-brown in colour with darker lines running through it, strongly scented and fairly hard.

* 1 anna = one sixteenth of a rupee, or approximately 2 cents.

It is a durable timber as compared with other coniferous woods. When under cover it will last to 20 years and upwards, while its life in the form of untreated sleepers may be taken at from 10 to 12 years. Weight, 35 lbs. per cubic foot. Two sets of tests carried out gave the average coefficient of transverse strength as 4.72 and 4.14 tons per square inch, respectively. The timber seasons well and fairly quickly, while owing to the tar oil in the wood it does not become water-logged during floating.

Uses.

The primary use of the timber is for railway sleepers. It is also used for bridge construction, and for house building in Jaunsar and elsewhere, as beams, rafters, door and window frames, boards and shingles. It is also used for boxes, packing cases, camp furniture and in general carpentry. Troup states that it is used for telegraph poles on the North West Frontier, elsewhere for masts, oars, well-construction, beer-vats and by the Ordnance Department for barrow wheels, transport boxes, etc. A Calcutta firm has pronounced the wood suitable for pencils, and that it is even better for this than *Podocarpus neriifolia.*

Outturn.

The total annual outturn of Deodar is approximately 4,837,000 cubic feet, of which over 3,000,000 cubic feet come from the forests of Kashmir and 900,000 cubic feet from Chamba State, and the rest from Government forests and private lands of the United Provinces and Punjab.

Prices.

Deodar broad-gauge* sleepers fetch Rs. 4-0-0 to Rs. 4-4-0 each, containing 3.3 cubic feet. At the depôts on the Jhelum River good logs fetch Rs. 1-2-0 per cubic foot and second-class about 11 annas per cubic foot; similarly, on the Chenab River, good logs fetch Rs. 1/ per cubic foot and those of poor classes 10 to 11 annas per cubic foot.

PYINKADO.

Distribution.

Xylia dolabriformis, or Burmese iron wood, is after teak the most important tree of Burma. It occurs in Burma and also on the Western and Eastern Ghats of India, generally associated

* Broad-gauge sleeper = 9′ 6″ x 10″ x 5″.

with teak. There is some doubt as to whether the Indian variety is botanically the same species as that found in Burma; anyhow, from the point of view of the timber, the Indian wood is far inferior to that of Burma.

Growth.

Pyinkado is a very common tree in most forests and grows with a straight bole up to 90 to 100 feet in height, and in favourable localities attains a girth of 9 to 12 feet at breast height.

Physical and Mechanical Properties.

The timber is dark red-brown, mottled and spotted; very hard and difficult to saw, especially seasoned timber; close grained, somewhat twisted; and cuts to a smooth, shiny surface. It seasons slowly, but not so slowly as Sâl, and cracks while drying, especially the Indian-grown variety. As regards durability, there is a great difference between the Burman and Indian timber. The former is extremely durable, lasting for 20 years and upwards as an untreated sleeper, while the Indian timber lasts only from 6 to 11 years, according to locality. It is a very heavy timber, weighing as much as 60 lbs. to the cubic foot. Gamble gives its transverse strength as varying from 7.68 to 9.57 tons per square inch for Burma Pyinkado and 3.23 to 6.07 tons per square inch for the Indian variety.

Uses.

The primary use to which the Burman timber is put is for sleepers, and owing to its strength, hardness and durability, it is admirably suited for that purpose. The Indian timber has also been used for sleepers but not to the same extent as the timber from Burma. It is extensively used in both Peninsulas for house building, in the form of posts, beams and scantlings; in bridge construction; and for buffers of railway trucks. In Burma it is used for telegraph posts, boats, dugouts and canoes, and also for ship building, especially as knees, crooks and keels (Troup). It is used for agricultural implements, such as ploughs, harrows, clod-breakers, oil presses and yokes; and elsewhere for tent pegs and railway keys; by the Ordnance Department for cart-poles and axle-cases; and for paving blocks in Rangoon. In Northern Kanara and Belgaum it is largely exploited as fuel to supply the Madras and South Marhatta Railway, up to 45,000 tons annually.

Outturn.

A computation was made some years ago in Burma in order to ascertain the possible outturn of Pyinkado sleepers per annum, with the result that 5,390,000 trees of over 7′-6″ girth, in other words mature trees, were found to be available within easy reach of the railway. It is estimated that these trees would give about 646,800 meter-gauge* sleepers per annum for 25 years.

The annual outturn of *Xylia dolabriformis* from the three Kanara Divisions and the Belgaum Division of the Bombay Presidency on the West Coast is put at 40,000 tons per annum, and that of Malabar, in Madras, at 50,000 cubic feet per annum.

Prices.

The price of Pyinkado in Burma is rising year by year and is likely to continue doing so, even more rapidly, probably, than that of teak. It now fetches Rs. 80 to Rs. 90 per ton of 50 c. ft. in Rangoon as scantlings and planks. The Burma Railways pay only Rs. 1-12-0 per meter-gauge sleeper (1½ c. ft.) so that it is a much more profitable business to convert it into constructional timber. On the West Coast, 'Pyinkado' or 'Jamba', as it is called in that locality, fetches from Rs. 24 to Rs. 32 per ton.

'IN', 'GURJAN' AND 'KANYIN'.

Dipterocarp species. For the sake of brevity, it is proposed to deal with the three species of Dipterocarps together. Three important Dipterocarps are found in Burma and the Andamans; namely, *Dipterocarpus tuberculatus,* known as 'In'; *Dipterocarpus turbinatus,* 'Gurjan'; and *Dipterocarpus alatus,* 'Kanyin'. The timbers of all three species are very alike and difficult to distinguish, so that the somewhat loose commercial term of 'Gurjan' timber is applied to all three. If anything, 'In' is superior to both the other two and 'Kanyin' superior to 'Gurjan'; so that anyone proposing to purchase 'Gurjan' and, instead, being supplied with either 'In' or 'Kanyin' would not be a loser.

Distribution.

'In' is found throughout Burma, often forming nearly pure forests. 'Gurjan' is found in the tropical forests of Burma, in Chittagong and in the Andamans; while 'Kanyin' is found in

* Meter-gauge sleeper = 6′ x 8″ x 4½″.

the moist forests of Bhamo, the Pegu Yomas, Arakan, Martaban down to Tenasserim, and in the Andamans.

Growth.

All three species grow to a large size, with fine, straight, clean, cylindrical boles, measuring 50 and 60 feet from the ground to the first branch. 'In' rarely exceeds 100 feet in height and 12 feet girth, while 'Gurjan' grows from 150 to 200 feet in height with a 15- to 18-foot girth. 'Kanyin' grows to a larger size than 'In', but is rarely found exceeding that of 'Gurjan'.

Physical and Mechanical Properties.

In colour, all three species may be described as red-brown, with large regular pores. 'In' and 'Kanyin' are hard and 'Gurjan' is moderately hard. They all season fairly well in the log, though very liable to split if the timber is converted green. As these three timbers have been described together, it will simplify matters to tabulate their mechanical properties:

Species	Air dry weight per c. ft., in lbs.	Strength to withstand transverse strain, in tons per square inch	Strength to withstand shearing, in tons per square inch	Strength to withstand compression, in tons per square inch	Remarks
Dipterocarpus tuberculatus					
'In'	50	5.3	0.65	2.97	Everett's tests in 1908
Dipterocarpus alatus					
'Kanyin'	50	5.9	------	------	Gamble
Dipterocarpus turbinatus		6.44	0.52	2.46	Burman timber
'Gurjan'	50	7.13	0.48	2.20	Andaman "

Uses.

'In', 'Kanyin' and 'Gurjan' are used for house building, chiefly in the form of scantling, rafters and planks; for boat building, furniture, joinery; and recently for railway sleepers, after treatment. Considerable quantities of these timbers are exported from Burma to Calcutta and the East Coast ports, and small quantities find their way to Europe.

Outturn.

The total outturn of these three species, together, is very considerable. The annual outturn during the last 5 years of

Dipterocarpus tuberculatus, 'In', from Burma, has been over 80,000 tons. The latest estimates put the sustained annual yield at 65,000 tons. The annual outturn of *Dipterocarpus alatus*, 'Kanyin', is put at 40,000 tons,—this is probably a conservative figure,—while the supply of *Dipterocarpus turbinatus*, 'Gurjan', from the Andamans, is stated to be 30,000 tons per annum. The total outturn of these three *Dipterocarps*, therefore, works out to 135,000 tons per annum from Burma alone.

Prices.

The price of these *Dipterocarp* timbers compares favourably with that of teak, and is even higher than that of Sâl. 'In' sells for about Rs. 70 to Rs. 75 per ton and 'Kanyin' for Rs. 60/—to Rs. 70/—per ton in Rangoon, while 'Gurjan' from the Andamans is valued at Rs. 60/—to Rs. 75/—in Calcutta. The above prices are for converted material, in the shape of boards, planks and rafters.

ANDAMAN PADAUK.

Distribution.

Petrocarpus dalbergioides, or Padauk, is confined to the Andamans.

Growth.

Padauk grows to a large tree, 80 to 125 feet in height, with a clear bole of from 20 to 50 feet, and a girth of 10 feet. The trunk is frequently much buttressed, especially when growing in damp localities or on low-lying ground.

Physical and Mechanical Properties.

Andaman Padauk yields a valuable ornamental timber of varying shades, from deep crimson to cherry red, pink reddish brown, and even dull light brown, the latter being known as "off-coloured". The timber is extremely hard, and difficult to bring to a fine polish, which when obtained, however, is lasting. Weight, 48 lbs. per cubic foot, and transverse strength, 5.01 tons per square inch.

Uses.

It is used by the Ordnance Department for constructing gun-carriage wheels. It is admirably suited for constructional work in all departments. It has given good results as a sleeper wood; its price, however, restricts its use to constructional and

ornamental work, such as for building purposes, internal fittings of Pullman cars, panelling, high-class furniture, etc.

Outturn.

The possible annual outturn of this timber is from 4000 to 6000 tons from the South and Middle Islands, while that from the Northern Island, an area which at present is not worked, may be put at 10,000 tons.

Prices.

The price varies with the colour; fine-coloured Padauk fetches Rs. 90/—per ton, f. o. b. Port Blair; average logs fetch Rs. 70/—per ton and "off-coloured" logs Rs. 55/—to Rs. 60/—per ton.

OTHER SPECIES.

In the first part of this paper it was stated that though eight of the Indian timbers are classed as of exceptional value, others exist which are also of very considerable importance. It is here not possible to give detailed accounts of each of these timbers, though the following should be included in any list of Indian timbers which are, or may in the near future be found to be of value to the engineer:

(1) *Acacia arabica*, (2) *Albizzia Lebbeck*, (3) *Anogeissus latifolia*, (4) *Artocarpus chaplasha*, (5) *Bischofia javanica*, (6) *Calophyllum tomentosum*, (7) *Dalbergia latifolia*, (8) *Dalbergia Sissoo*, (9) *Dipterocarpus pilosus*, (10) *Grewia tiliaefolia*, (11) *Heritiera minor*, (12) *Homalium tomentosum*, (13) *Hopea odorata*, (14) *Hopea parviflora*, (15) *Lagerstroemia microcarpa*, (16) *Mesua ferrea*, (17) *Myristica Irya*, (18) *Pinus excelsa*, (19) *Pinus longifolia*, (20) *Pterocarpus macrocarpus*, (21) *Terminalia tomentosa*, (22) *Terminalia Myriocarpa* and (23) *Terminalia paniculata*.

III. WHAT IS BEING DONE TO MAINTAIN THE SUPPLY OF TIMBER IN INDIA.

Introduction of Forestry in India.

As long ago as 1825, the question of preserving the valuable forests of India came under consideration. The initial start was made in Madras when the famous Nilambur teak plantation was created by Mr. Conolly, a distinguished member of the Indian Civil Service. A definite policy was, however, not inaugurated

till 1856, when Lord Dalhousie, the then Governor-General of India, laid down rules and regulations with a view of affording wide-spread protection to the valuable forests both of India and Burma.

Forest Staff.

The next step taken was to create a Forest Service, which, though small to commence with, now consists of an Inspector General, 2 Chief Conservators, 23 Conservators, a President of the Forest Research Institute and College, 16 Research Officers and Instructors, and 195 Deputy and Assistant Conservators,—the above officers all having been trained in Europe,—and 202 Extra Deputy and Assistant Conservators of Forests who have been trained in India. The Rangers, Foresters and Guards, a fair percentage of whom are trained men, bring the strength of the Department up to about 16,000.

Area of State Forests.

The first step taken after creating a special Forest Service was to demarcate, settle and finally map the State Forests, which now cover 243,500 square miles, or approximately one quarter of the whole of British India.

Working Plans.

Soon after the forests had been placed on a legal basis and as the forest maps became available, the energies of the Department were directed toward making "working plans". Most of the plans for Teak and Sâl forests have been drawn up on the "selection system"; others, where the forests were in a damaged state when taken over, are being provisionally treated under "improvement fellings"; while in places where the local conditions are less favourable to forest growth, the "coppice", with "standard system", has been introduced. In some Provinces, the "working plans" are practically completed; in others, much work still remains to be done. The total area under regulated plans now amounts to over 45,000 square miles.

Plantation.

In order to increase the growing stock, extensive plantations have been created, chiefly of Teak in Burma and Madras, of Casuarina on the East and West Coasts, of Deodar in the Western Himalayas, of Rubber in Assam and Mergui, a large irrigated plantation of Sissum and other species in the Punjab plains, and

of Eucalyptus in the Nilgiris. The area under plantations now amounts to 148,617 acres.

Protection from Fire, Grazing and Theft.

Fire conservancy and protection from theft have been strictly enforced; grazing is regulated and, where possible, restricted to grazing as against browsing; while areas under regeneration are completely closed to grazing of any sort for at least ten years.

IV. THE ECONOMIC POSITION OF THE TIMBER TRADE IN INDIA.

Indian Requirements.

In a country such as India, with a population of over three hundred million, the amount of timber required is necessarily very large. By far the greater quantity of timber is obtained from the Government forests, amounting to approximately 75,000,000 cubic feet of timber and 182,000,000 cubic feet of fuel per annum. Estimating the outturn of timber from the forests other than those belonging to the State to be 25 million cubic feet, the consumption per head of population works out to under one third of a cubic foot, which, compared with European countries, is a low figure. The reason for this is doubtless due to the fact that by far the greater portion of the population is agricultural, living in poor-class houses, made of sun-dried mud bricks, with a roof consisting of small rafters and thatch or tiles; the total amount of timber used in the construction of which is extremely small, while the number of persons living in one small house is comparatively great. Again, the timber used is often Teak, Sâl or Deodar, all extremely durable timbers, and requiring to be renewed only after long intervals.

As has been explained above, the demand for timber in India, when taking into consideration its enormous population, is relatively small when calculated on a basis of consumption per head of population; nevertheless, the indigenous supply is barely sufficient to meet all requirements, a point which is clearly demonstrated by the fact that imports of timber are increasing, while the prices of all classes of indigenous timber have been and are still steadily rising. Thus, the price of Deodar sleepers has advanced from Rs. 2/12/—in 1893 to Rs. 4/4/—in 1913, and con-

structional timber has advanced in like proportion. Best Burma teak could be purchased in the log (round) at Rs. 70/—per ton 20 years ago, while now selected logs fetch as much as Rs. 120/— to Rs. 150/—per ton, and in exceptional cases Rs. 200/—.

Another indication that the supply can only just meet the demand, and probably a more convincing proof of this state of affairs than that shown by the increase in the cost of timber, is that certain species which formerly were rejected as useless by the local inhabitants are now readily purchased by them for constructional purposes.

Imports and Exports.

When reviewing the economic position of any particular article of commerce, it is necessary to study the figures of imports and exports. The most recent figures available show that about 18,000 tons of teak wood are imported into India, chiefly from Siam and Java; about 35,000 tons of Deal and Pine wood, chiefly from America; 40,000 tons of other timber, chiefly Dipterocarps, from the State Settlements; and Jarrah from Australia. Against this, some 60,000 tons of teak are exported from Burma and India, which chiefly find their way to the United Kingdom, Germany, Ceylon, Japan and the Cape; and of other timber, about 7,000 tons, chiefly Blackwood, Padauk, and Sandalwood, which find their way to Europe. The excess of imports over exports amounts to about 26,000 tons, so that India may be said to be not absolutely self-contained as far as timber is concerned.

Conclusions.

The chief factors governing the economic position of the timber trade in India are the progress of civilization, the rate of increase of population and the measures being taken to keep up the supply of timber. India is fast absorbing Western civilization; it is advancing with long strides in education; the increase in population is rapid; and development in engineering is fully keeping pace with the times; so that there can be no question of doubt that the demand for timber will increase by leaps and bounds. It becomes a nice problem as to whether the forests will be able to stand the strain. Here, two factors have to be considered; namely, that the majority of forests were in a very bad condition, owing to unregulated fellings, when pro-

tection was first introduced some five years ago and have not yet had time to recover; and, secondly, that the forests contain a very large number of species the value of which has not as yet been realized in the timber markets. Time will cure the first defect, and when the forests come into complete bearing the out-turn will be more than doubled; while the question of utilizing greater numbers of the available species is automatically adjusting itself as the market value of the more valuable species becomes prohibitive; and, lastly, a factor which will tend to increase the supply in the future is the development of roads and mechanical traction, especially in more remote forests. It is thought that in spite of the steady increase in demand which is sure to arise, that, anyhow, for some years to come, the State forests will be able to cope with the ever-increasing demand, provided that strict protection be maintained, that every endeavour be made to utilize to the utmost the many varieties of timber available, that a liberal policy of allotting funds for the construction of new roads, tram lines, wire-ways, improving floating streams, etc., be adopted and that the staff be increased in proportion to the development which will take place under a forward policy.

Paper No. 98

TIMBER OF RUSSIA.

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FOREST AREAS.

The forests in Russia cover an area of 1355 million acres, that is to say, they exceed 2½ times the forest area of the United States of America, and are distributed among the separate parts of the empire in the following way:

Forest-land Area in Millions of Acres.

For European Russia	446
Finland	38
The Caucasus	18
Asiatic Russia	853

It comes to about 8 acres of forest land to one inhabitant. The forest land on the average constitutes 37% of land area. This percentage of forest area varies a good deal in different provinces.

In Finland the forest area amounts to 45%. Within the bounds of European Russia the percentage of forest area is particularly high in 8 of the provinces: Wologda 82%, Perm 57%, Nowgorod 54%, Petrograd 50%, Olonetz 41%, Kostroma 47%, Archangel 45%, Viatka 41%. It changes oftener in the middle region of Russia from 20 to 30%. On the borders of the Vistula the average forest land is below 20%. In the south steppe governments and southeast Wolga provinces the percentage of the forest area varies between 1 and 1½; the government of Cherson up to 10%; Padol 1%; the average forest land in the Caucasus constitutes 16%, with variations beginning at 77%; the Black Sea province, Stavropol, up to 1%.

FOREST OWNERSHIP.

According to forest ownership, the Russian forests are divided into government forests, private forests and those belonging to all kinds of establishments. Government forests in five northern provinces—Archangel, Wologda, Perm, Olonetz and Viatka—amount to 198 million of acres, or 83% of the whole area of this region. In the remaining part of European Russia, excepting the Vistula region, the national forests amount to 27 million acres, or 22% of the whole extent of forests. In the Vistula country there are about 2 million acres, or one-third of the whole space occupied by national forests. In the Caucasus the national forests occupy 8 million acres, or 45% of the whole area. In Asiatic Russia national forests comprise nearly 772 million acres, which amounts to 90% of the whole area.

THE FORESTS OF THE EXTREME NORTH OF EUROPEAN RUSSIA.

Distribution of Species.

South of the strip of treeless, marshy plain (so-called tundra), adjacent on the north to the Frozen Ocean, in Finland, European Russia, the forests of conifers contain two chief species, the spruce (Picea excelsa) and the Scotch pine (Pinus silvestris). On sandy soils grow pure pine forests; on clayish fresh soils, a mixed forest of Scotch pine and spruce. After the cutting out of the Scotch pine in the mixed forests, the spruce often exchanges place with the Scotch pine, and the latter returns to its former territory only after fires have destroyed all the spruces, sparing only the seed-bearing trees of the Scotch pine. On the east side of the northern region of the conifers, between the Onega River, Nigni-Nowgorod and Ufa, with soils rich in calcium, we meet with Siberian larch (Larix sibirica) usually mixed with Scotch pine and spruce. Within the limits of the growing of Siberian larch, there happens to be likewise the Siberian fir (Abies sibirica).

On the extreme northeast, in the sector, the bow of which attains the 65th parallel and in the south descends to the 57th parallel, the Siberian pine is met with.

Of the hardwoods, the aspen and birch trees extend into the northern parts, but on going further to the north, the

aspen becomes more and more scarce. The hardwoods make their appearance after the clearing out of the conifers or after fires; at the beginning of their history of life, the hardwoods grow with the conifers, dying out 100 to 200 years sooner.

Yield.

In the government of Petrograd, on the best soils the pure forests of Scotch pine and spruce give the following yield:

Species	Age Years	Height Feet	Diameter breast-high Inches	Total volume per acre Cubic Feet
Scotch pine..........	100	88	12	7075
Spruce	100	90	12	7650

Notwithstanding the severe climatic conditions of the government of Wologda, in the district of Solvytchegodsk the pure spruce forests at the age of 200 years (82 feet high and 14 inches in diameter, breast-high) gave 5300 cubic feet to the acre, while the largest separate trunks hardly reached 121 feet in height and 38 inches in diameter, breast-high.

The Siberian larch (Larix sibirica) in the government of Archangel, on a clayish soil with a dolomite sub-soil, reaches at the age of 200 years 108 feet in height and a diameter of 17 inches. A mixed larch-pine-spruce forest on such soil gives in 160 years 6500 cubic feet to the acre.

Mechanical Properties and Commercial Uses.

In consequence of the comparative of growth, the trees of the northern forests have narrow annual rings and are therefore remarkable for high technical qualities.

The technological laboratory of the Imperial Forest Institute, under the direction of Prof. H. A. Phillipoff, has been making most divers investigations of Russian woods, and states that of the conifers growing in the governments of Wologda and Kostroma, the Scotch pine timber 100 to 200 years old in air-dry condition, with 10 to 12% of moisture, has a compressive strength parallel to grain of 525 to 600 kg. to the square centimetre (7450 to 8525 lbs. per sq. in.). The greatest compressive strength—740 kilograms per square centimetre at a specific gravity, air-dry, of 0.573—is noted as

belonging to the Scotch-pine woods in the government of Viatka.

The well-renowned Russian pine is used for lags, square beams, boards, masts, shingles, railroad ties, etc.

The spruce attains the best mechanical properties as resonance material for musical instruments, in the government of Viatka; it has a specific gravity, air-dry, of 0.496 and a compressive strength parallel to the grain of 630 kg. per square centimetre (8960 lbs. per sq. in.).

During the first period of lumbering in the north there was a prejudice against the spruce, and the price of the spruce was one half that of pine timber. At present, spruce timber, on the average, is priced at 10% below that of pine, but each year the reputation of the spruce rises. The Northern and Perm railroads use the spruce for ties. The Perm railroads laid down in the year 1912 over one-half million spruce ties. Not long ago the official instructions of the Department of Means of Communication demanded that the spruce ties should not be used longer than 4 years; whereas experience has proved that they can serve from 5 to 6 years.

The wood of the Siberian larch (Larix sibirica) is distinguished by its first-rate qualities. Thanks to its strength and elasticity, it is superior to all conifers. Its compressive strength parallel to grain at a specific gravity, air-dry, of 0.530-0.690, varies from 590 to 800 kilograms per square centimetre (8380 to 11,350 lb. per sq. in.), and in some samples from the government of Viatka, even 900 kilograms to the square centimetre (12,780 lb. per sq. in.). Its modulus of elasticity in bending comes to 112 to 145 tons per square centimetre (3,180,000 to 4,120,000 lb. per sq. in.). The hardness, by the methods of Brinell and Tanka, of the larch timber is, in general, not high, namely 350 to 450 kilograms per square centimetre (4960 to 6390 lb. per sq. in.); that is why it is easily worked with tools. The pine is a valuable material for underground constructions and railroad bridges, is used for railroad ties, and likewise for ship building.

The Siberian pine (Pinus Cembra) wood is soft and, comparatively, not of a very high compressive strength parallel to the grain (up to 400 kg. per sq. cm. = 5675 lb. per sq. in.),

but thanks to its pleasing design, it is used readily for many purposes.

The qualities of the Siberian fir (Abies sibirica) timber are not so good as those of the spruce, having a specific gravity, air-dry, up to 0.421 and a compressive strength parallel to the grain equal to 485 kg. per sq. cm. (6880 lb. per sq. in.).

Lumbering and Management.

In the northern governments of European Russia up to now there has been a demand chiefly for saw-logs, the predominating dimensions of which at present are 23 1/3 feet in length and 8¾ inches diameter at the top. These dimensions are acquired by the trees of the northern forests in 120 to 200 years. In connection with such demands of the market, the single-tree method of cutting was employed, by which the superior logs were not utilised; only of later years have they begun at times to utilise the clean strip method, and make use of the tops and thin trees for pulp and paper and other productions. The stumpage price in the northern forests varies; for Scotch pine it is from 2 to 5 cents and for spruce from 1 to 3½ cents per cubic foot.

In the wide-spread river system—with its two chief arteries, the Northern Dwina and Pechora, flowing into the Frozen Ocean—the quiet flow of the rivers, owing to the even nature of the country, makes the transport of wood cargoes very easy. In general, it is supposed that in the governments of Archangel and Wologda no merchantable forest is farther than ten miles from a floating-down river. Owing to this, the northern timber forms the chief object of Russian export. The importance of the northern forests may be judged by the circumstance that Russia transports 4% of all the timber to Great Britain, whither the merchandise is chiefly directed from the harbours of the White Sea—Archangel, Onega, Keme, Mesen, Corok, Suma, Keretikawda and Umba.

In 1912 out of the national forests in five northern governments (Archangel, Wologda, Olonetz, Perm, Viatka), there was designated to be cut 942,480 thousand cubic feet of live timber; whereas in reality there was sold and given for free use 398,972 thousand cubic feet, or 40%. The correlation of these data points out the unprofitable economic conditions

which abound in the northern forests. But with all that, it must be noted that the northern pine up to now has often been fixed upon to be chopped out in larger quantities than was necessary, according to its history of life and rate of growth. The reckoning upon the use to be made of the northern forests proceeded mostly on the supposition that, with the spruce, as well as with the pine, upon the chopping out of the trees of merchantable size from the uneven-aged selection forests, the trees growing up would be ripe for the next cutting in from 60 to 80 years. In reality, in mixed forests on fresh soils we have even-aged pine trees and the select cutting-out often adopts the character of one clear cutting, in which case, for the northern pine, rotations of 140 to 160 years ought to be fixed. Only unfavourable economic conditions smooth down a certain irregularity of this calculation.

THE FORESTS OF THE OTHER PROVINCES OF EUROPEAN RUSSIA.

Distribution of Species.

The forests of the other governments of European Russia, to the south of the line joining Petrograd with Ufa, are located in the country where the conditions of the climate are milder. Among the trees growing together in the forest, appear broad-leaved kinds, although conifers predominate to a considerable extent. So that the southern border of purely pine forests of European Russia goes to the 50th parallel in the west, rises in the middle (government of Moscow) to the 55th parallel and in the east (government of Orenburgh) descends to the 51st parallel. The southern border of the spruce forests to the west and in the centre goes nearly on a parallel with the above border, the spruce receding only a little from the latter, and toward the east goes over into the Siberian forest considerably to the north of the pine—about the 55th parallel.

To the west of the meridian of Riga and Grodno there are forests containing European larch (Larix europoea) and European fir (Abies europica). In the mountainous western part of the Vistula the European fir forms, on large areas, beautiful clean forests, sometimes with a mixture of European beech (Fagus silvatica), and these forests are remarkable for their quick growth.

Of the soft, leafy trees, the birch and aspen, throughout the whole region to the very steppes, accompany the conifers —on fresh soils making their appearance after cutting.

The alder (Alnus glutinosa) in central and southern Russia is met with chiefly on the banks of lakes, river bottoms and especially at the issue of iron springs. The northern border of valuable merchantable forests of oak (Quercus pedunculata) is approximately along the 56th parallel of latitude. In central and southern Russia the oak grows intermixed with ash, maple and elm; in the eastern part of Russia the basswood appears; in the southwest, the hornbeam. In the western part of the region we meet with another kind of oak—Quercus sessiliflora. The best oak forests are found in the governments of Minsk, Grodno and Podol, where in deep, clayish soil the oak in 220 years reaches the height of 120 feet, with a diameter of 30 inches, and half of its trunk free from branches, yielding often 6500 cubic feet of timber per acre.

The ash (Fraxinus excelsior) attains its best development within a space limited by a line going through the governments of Livonia, Courland, Vilna, Moghilef, Wladimir and the right-hand bank of the Wolga.

The best basswood forest (Tilia pardifolia) extends from the northeastern corner of the government of Kostroma to the south of the steppes; the lindens of the government of Ufa are particularly famous. In the south of Russia extends Tilia grandifolia.

Mechanical Properties and Commercial Uses.

Concerning the mechanical properties of the hardwoods, a more detailed investigation ought to be made of the oak, which has a remarkable importance in trade. The technical qualities of the oak are determined a good deal by the conditions of soil and location. On sandy soils rich in humus, in the government of Kief, the conditions are very favourable for the growth of more elastic timber, with a specific gravity, air-dry, of 0.689-0.730, a compressive strength parallel to the grain of 603 to 678 kg. per sq. cm. (8560 to 9650 lb. per sq. in.), and a hardness, by the method of Brinell and Tanka, of 520 to 620 kg. per sq. cm. (7390 to 8800 lb. per sq. in.). Such oak is used for construction timber, tight cooperage stock, boards, carpenter

work and so forth. The oak growing on the clayish soil of the Kazan province has a specific gravity of 0.658-0.737 and a compressive strength parallel to grain of 592 kilog. per square centimetre (8400 lb. per sq. in.); the hardness of such an oak is very considerable—up to 720 kilog. to a square centimetre (1000 lb. per sq. in.)—which is a reason why the use of this kind of oak is rather limited; it is unwillingly used for carpenter work. But for that, such timber does very well for inlaid floors. The modulus of elasticity of this oak is rather high, up to 122 tons per square centimetre (1,735,000 lb. per sq. in.); that is the reason why there is a great demand for it for wagon beams. The oak on the clayish soils in the government of Woronege is likewise distinguished for its great strength (up to 620 kilog. —8800 lb. per sq. in.) and elasticity (up to 150 tons—2,130,000 lb. per square centimetre); but its hardness, too, is very high.

In the government of Moghilef, on sandy soils, the oak produces rather narrow annual rings of soft timber, with a specific gravity of 0.600, a compressive strength of 543 kilog. (7700 lb. per sq. in.), a modulus of elasticity of 107 tons (1,-520,000 lb. per sq. in.), and a hardness of 551 kilog. (7837 lb. per sq. in.) per square centimetre. Such oaks are sent abroad as construction timber and cooperage staves. The oak from the government of Tula, growing likewise on clayish soil, has a specific gravity of 0.524-0.657, a somewhat lower compressive strength, 527 kilog. per square centimetre (7500 lb. per sq. in.), and a hardness of 598 kilog. (8500 lb.); the modulus of elasticity of such an oak is likewise not high—99 tons per square centimetre (1,410,000 lb. per sq. in.).

With such mechanical properties, the Tula oak is readily bought by carpenters; and due to its elasticity, it is used for the making of hoops.

The oak in the government of Cherson, on clayish soils with lime sub-soil, has a specific gravity of 0.580-0.690 and gives timber of middling qualities as to strength and hardness, and as Holland beams is sent abroad.

On river bottoms the trees of the open oak forest have solid trunks with wide annual rings, but the tree stems of such a forest grow more to the crown and the clear length is

large; that is the reason why these trees are chosen for construction purposes.

In Russia the dry sand and alkali lands present the most unfavorable conditions for the growth of oak. On the first, the oak has narrow annual rings and very porous wood; the specific gravity is 0.590, the compressive strength parallel to grain does not exceed 450 kilog. (6400 lb. per sq. in.), and the hardness is equal to 300 kilog. (4250 lb. per sq. in.) per square centimetre. On alkali soils the oak discloses a middling compressive strength (558 kilog. = 7940 lb. per sq. in.), a low modulus of elasticity (93.5 tons = 1,330,000 lb. per sq. in.), but very great hardness.

Among other hardwoods growing in European Russia, the timber of the ash is used as a building material and for other kinds of work; it is used for cars, furniture, agricultural machines and implements, carpenter work, oars and hoops, and is used, in general in those cases where hickory is used in America.

In some places (for instance, in the government of Woronege) the lime is used in buildings, which even after the lapse of 50 to 60 years do not require repair; it is used also for the making of trunks, boxes, bee-hives and so forth.

The aspen, once considered a weed-tree, is at present often sold at a price equal to that for the spruce. The cylindriform elastic timber of the aspen possesses a considerable compressive strength—548 kilog. per square centimetre (7800 lb. per sq. in.). It is willingly used for the construction of peasants' cottages (in the government of Tula, and elsewhere) and is good for underground erections; out of it are made roof shingles, cooperage stock and cars. For match, pulp and paper fabrication there was exported abroad during two years (1911 and 1912) from Petrograd alone 4¾ million cubic feet of aspen timber. An experiment was made at the Riazan station, on the Riazan-Uratian Railroad, of putting down aspen ties, and after a service of 6 years the ties were not yet worn out.

The wide-spread birch in European Russia is in great demand in the life of the Russians, in the guise of a first-rate fuel and as material for small-work business. The scientific investigations, in the laboratory of the Imperial Technical Society,

of the birch-wood samples, dried to a constant weight, confirm the usefulness of the Russian birch-wood as fuel. The following table shows the results of these investigations:

Species	Calorimeter coefficient	Specific gravity, dry wood	Specific Calorimeter coefficient as obtained by multiplying calorimeter coefficient by specific gravity
Birch	4968	0.570	2832
Alder	5047	0.436	2200
Scotch pine	4929	0.421	2075
Spruce	4857	0.384	1865
Aspen	4953	0.374	1542

The mechanical properties of the birch are very considerable: at a specific gravity of 0.640, its compressive strength is about 800 kilog. per square centimetre (11,380 lb. per sq. in.), but in European Russia the birch is not used for building purposes on account of a great amount of large stems, as on durable timber belonging to other species.

The alder offers good material for underground water-pipes, cylinders for pumps, cigar-boxes, and all kinds of carpenter work.

Strong, elastic, very tough timber of three kinds of elm (Ulmus campestris, Ulmus montana and Ulmus effusa) is used for carriages, naves of wheels, wind-mill rollers and wheels, pulleys, screws, and other parts of machines, water-pipes, boats and so on.

The maple (Acer platanoides and Acer pseudo-platanus) is adopted chiefly for carpenters' and turners' work, for the making of carriages and musical instruments.

The dense, tenacious timber of the hornbeam is used for farm implements, handles for instruments, turnery and, likewise, fuel.

Lumbering and Management.

It is clear that within the limits of the immense Russian territory, between Petrograd and Ufa nearly to the Crimea and the Caucasus, the stumpage prices for timber will be as different as the economic condition and character of the forests. The highest prices reign in the regions of the better quality of each species of timber, in the presence of favourable economic surroundings, or in regions wanting in timber. The highest

stumpage prices for timber of the chief commercial species in the national forests are as follows:

Species	Stumpage price in cents per cu. ft. of saw-logs	Province
Oak	24	Grodno
Scotch pine	13	Plotzk; Warsaw
Spruce	11	Petrokoff and Colish
Birch	11	Cherson
Aspen	9	Orel

The real timber sale prices often exceed greatly the fixed stumpage prices. On the contrary, low prices are observed either in economical, out-of-the-way places, or in a range beyond the limits of the best development of the said species. The lowest stumpage prices are given below:

Species	Stumpage prices in cents per cu. ft. for saw-logs	Province
Oak	1	Twer
Scotch pine	1	Orenburgh
Spruce	¾	Moghilef and Kazan
Birch	⅓	Nigni-Nowgorod
Aspen	⅓	Nigni-Nowgorod and Kostroma

The rivers Wisloka, Niemen, and western Dwina serve for exporting timber abroad and to the surrounding countries. Riga, Libau, Windau and Petrograd, with Kronstadt, are some of the ports of the Baltic Sea from which timber is shipped abroad. On the Wolga and Dnieper, timber is floated down to forestless provinces where, due to climatic conditions and in consequence of the extreme richness of the black earth soils, forests grow only in dells, ravines and similar low places, or in sandy, alluvial soils. From the Black Sea ports, as Odessa, Nikolaief, Cherson, Noworossiesk and Batum, timber is exported abroad.

As to the character of the national forest regulation, the predominating rotation of cutting is usually 120 years for pine, 100 years for spruce, 100 years for high-oak forest, and 60 years for coppice sproat-oak forest and soft, leafy trees. In the greater part of the national forest the clear system of cutting is adopted, by close adjoining strips to be felled; but

for the cutting of pine, the trees to be felled are sometimes divided into alternate strips. It is seldom that for spruce forests with a mixture of hardwood a gradual cutting, or so-called compartment system, is laid out. The width of cutting strips for pine, spruce and oak varies from 140 to 210 feet; for other hardwoods it amounts to 350 feet. For the reproduction of pine on the woods to be felled, about 10 seed-bearing trees are left to the acre, or seeds and natural seedlings are expected from the adjoining strips of the old forest. In spruce and oak forests, with favourable economic conditions, planting systems are adopted; in thinly populated regions, natural reproduction is allowed to take place, in which cutting strips of the spruce are overgrown at first with hardwood, but after a certain lapse of time the spruce begins to reappear; in cutting strips from which oak trees have been felled, there appears an undergrowth of oak, in the company of other hardwood species.

In the mountainous forests of Crimea which have protection and a water-sheltered character, the trees are chiefly commercial; the Crimean pine (Pinus laricio Pallassiano), growing on limestone, reaches in 200 years to the height of 85 feet with a diameter of 28 inches and forms 6500 cubic feet of timber per acre. The durable wood of the Crimean pine is richer in resinous products, is denser and heavier, than that of the usual Scotch pine.

In all the governments of European Russia, excepting five of the northern ones (Archangel, Wologda, Olonetz, Perm and Viatka) it was decided in the year 1912, to cut only 1,318,064 thousand cubic feet of live timber from the national forests; in reality there was sold and given for free use 1,066,786 thousand cubic feet, or 81%.

THE FORESTS OF THE CAUCASUS.

Distribution of Species.

The steppes coming out of south Russia into the Caucasus in the Kuban region, with the change in the topography of the country and changes in climate, are diversified in forests, the composition of which is very varying. There is not one region in the limits of the Russian empire that is as rich in timber

species as the Caucasus. It is reckoned that there are about 300 kinds of trees and bushes. Among 100 of solely timber species, the most remarkable are: of conifers—six species of pine, the eastern spruce, Caucasian fir and yew tree; of hardwoods—Caucasian beech, the eatable chestnut, 11 species of maple, two species of hornbeam, black walnut, the Caucasian palm, iron-tree and others. The hardwood species occupy three quarters and the conifers one quarter of the whole forest area. Nearer to the steppes, the lower and drier slopes of the northern Caucasus are covered with hardwood of oak (Quercus sessiliflora), ash, elm and maple. With the increase in elevation above sea-level and the increase in moisture, the oak is replaced by the Caucasian beech (Fagus silvatica orientalis), the greatest development of which is reached on slopes at a height of 3000 to 4000 feet above sea-level. In the western part of the mountainous region, where the mountains are not high, the northern slopes of the Caucasus are covered with these trees to the top. On moving to the east, at an elevation of 4000 to 6000 feet appear luxurious pine forests, chiefly Pinus silvestris, Caucasian fir (Abies Nordmanniana) and spruce (Picea orientalis), sometimes with a mixture of yew trees (Taxus baccata). The spruce and pine grow up to 7000 to 8000 feet above the level of the sea, where these species often fill up the forest line. The farther to the east we go the scarcer become the spruce and Caucasian fir, but the pine is more abundant. There are especially large pine forests on the slopes of the Elburz Mountains. To the east of the Elburz, among conifers the pine and juniper are only to be found.

The Black Sea coast of the south branchings of the Caucasus Mountains is remarkable for a luxurious development of timber growth such as is seldom met with in other countries of the globe. This strip puts one in mind of the Pacific Coast of the United States of America. Here (in Batum) a precipitation of 90 inches falls yearly, and the temperature in extremely cold winters falls only to 17° F. Some of the trees here in from 5 to 8 years' time attain a height of 35 to 50 feet. The Caucasian beech grows here with a mixture of chestnut (Castanea vesca), likewise the oak, at a height of 2000 to 3000 feet; approximately on this line grows a very valuable, so-called

Caucasian palm (Buxus sempervirens); at higher elevations appear spruce forests, with a mixture of elm, maple and other trees.

Also, excessively favourable conditions for the growing of timber are observed in the east corner of the south Caucasus on the west coast of the Caspian Sea, in the surroundings of Lencoran. Here in low places grow the oak (Quercus castanefolia), iron-tree (Parrotia persica), Caucasian palm (Buxus sempervirens) and alder (Alnus cardifolia and A. glutinosa). Higher up in the mountains the forests consist of ash (Fraxinus excelsior), oak (Quercus castanefolia), circasian walnut (Juglans regia) and hornbeam (Carpinus Betulus). On a strip between 2000 and 5000 feet elevation we meet with the Caucasian beech.

The forest area in the Caucasus classified according to commercial species is as follows:

Species	Area Acres	In % of the general forest area
Caucasian beech	4,636,000	25.8
Oak	3,822,000	21.2
Hornbeam	2,364,000	13.1
Pine	1,496,000	8.3
Spruce	1,219,000	6.7
Fir	1,108,000	6.1
Other species	3,372,000	18.8
	18,017,000	100.

Yield.

Under present conditions the growth and yield of the chief species are as follows:

Species	Age Years	Height Feet	Diameter breast-high Feet	Total volume per acre Cubic Feet
Caucasian beech	200	140	7	8,000
Oak	120	100	2	5,000
Pine	100	120	4	8,000
Spruce	250	180	5	12,000
Fir	250	170	7	16,000

The Mechanical Properties and Commercial Uses.

The wood of the Caucasian beech possesses very high mechanical qualities: its specific gravity varies from 0.593 to 0.695; compressive strength parallel to grain from 610 to 740

kilog. per square centimetre (8670 to 10,525 lbs. per sq. in.); its modulus of rupture at breaking from 98 to 138 tons per square centimetre (1,393,900 to 1,963,000 lb. per sq. in.): hardness, by method of Brinell and Tanka, from 450 to 686 kilog. per square centimetre (6400 to 9757 lb. per sq. in.). The Caucasian beech gives superior wood for under-water constructions, staves for butter-barrels, railroad ties, cars and wagons, furniture, turning and other productions.

The oak wood stands out as an excellent construction material for building purposes, and is used in great quantities for railroad ties (the Wladicaucasian Railroad in 3 years, 1911 to 1913, has laid down about 2 million oak ties).

The chestnut produces very durable timber, particularly prized where the structures made of it are built in damp places; it is also used as timber for building. On the Caucasian railroads chestnut ties are laid down, as well as those of oak, without even any preservative treatment.

A very dense and heavy wood (specific gravity in air-dry condition 1.06), as hard as ivory and very durable is the Caucasian palm; it sells at a high price, and is sold by the pood for carpenters', turners' and sculpturing works, and especially for engraving. The Caucasian walnut has a highly merited reputation beyond Russia and is used for furniture, cabinet making, gun stocks and aeroplanes; the spunks are especially famous for a very handsome design. The wood of ash, lime and poplar is used for building and for small works. The elm is used for wagon and machine manufacturing. The utilization of other kinds of wood is the same as in European Russia.

As building material the Caucasian pine is foremost. As a distinction from the European spruce, which, compared with the pine, is less lasting and rots sooner, the Caucasian spruce lives as long as 400 years and remains whole for 200 to 250 years, therefore, it is to be expected that its use as timber for building will develop yearly.

The wood of the Caucasian fir has been, up to the present, less used for building than that of other conifers. The wood of the juniper (Juniperus foetidissima) is distinguished for its great durability, is not subjected to worm-eating in the course of 100 years and is prized highly for building purposes. A

very durable wood of another species of juniper (Juniperus excelsa) is likewise used for building, and under the name of cypress is used for pencils, crosses, turning and other kind of handiwork.

Lumbering and Management.

The stumpage prices for saw-logs of the Caucasian species in the national forests vary as follows:

Species	Stumpage price in cents per cubic foot
Caucasian beech	1½–5
Oak	1 –5
Pine	1½–8
Spruce	1 –6
Fir	1½–8

In the Caucasian forests, according to their mountainous character and wide-spread economical conditions, a method of single-tree cutting is adopted. Only of late has there been adopted, in some cases, a clean-strip and compartment system of cutting. The use of the primeval Caucasian forests up to the present has been very slight. Thus in the year 1912 it was decided that out of the national forests 164,736 thousand cubic feet of livewood was to be permitted to be cut. In reality there was sold and given for free use only 39,754 thousand cubic feet, or 1/5 of the intended quantity. To raise the productiveness of the Caucasian forests, it is necessary to spend money on the building of roads, chutes and manufactories for wood-working, and undoubtedly the sums thus spent will bring in a good percentage of profit, as the want of timber is felt not only in the steppe parts of southern Russia but also in the southern European dominions, to which the transport of Caucasian wood will be convenient and cheap.

FORESTS OF ASIATIC RUSSIA.

Distribution of Species.

The climatic conditions of Asiatic Russia are very diverse, beginning with the cold government of Takutsk, where during the winter the quicksilver freezes in the thermometer, and finishing with dry and hot Turkestan, similar to semi-arid New Mexico and Arizona. The character of the forests is under the influence of these different conditions of nature.

Siberia, occupying a space of above 5 million square miles to the west up to the Yenisei River, has a plain—to the east—of a mountainous character.

The lines of the forests adhering to the Ural ridge of the western frontier of Asiatic Russia remind one of the forests of the northeast districts; spruce (Picea abovata), fir (Abies sibirica), Siberian pine (Pinus Cembra) and larch (Larix sibirica) form the chief species of the government of Tobolsk. Among the soft leafy trees the birch (Betula alba) and the aspen (Populus tremula) predominate, but the oak and elm (Ulmus campestris) do not extend beyond the ridge of the Ural into the government of Tobolsk, the severe winters of which put a stop to the extension of these broad-leaved species.

The "black taigo", consisting of fir, spruce, Siberian pine, with a mixture of aspen and birch, are found on the soils containing alumina and iron oxides which have been washed down from clayish and sandy soils above. Pure pine forests stretch along sandy borders on the elevated right-hand banks of the rivers. The greater the number of rivers the oftener the forests of pure pine.

Fires make conifer forests exchange with "white taiga", consisting of birch and aspen.

On the Altai, the valleys of the rivers are occupied by birch and poplar (Populus laurifolia). Out of the hardwoods that we meet with here, the lime is one of the most tolerant members of the flora that existed in former geological periods in a warmer climate in Siberia and died out later on. The lower slopes of the mountains are covered with birch, spruce and pine, which do not rise higher than 1000 feet above sea-level. Forests of pure Siberian pine (Pinus Cembra) occupy at times considerable areas. In the government of Tobolsk there are areas exceeding 25,000 acres occupied at a stretch by this specimen of conifers.

The farther we go to the east, the country becoming more elevated and limestone outcroppings on the mountain slopes becoming frequent, the oftener we meet with the larch. In western Siberia there are two species of larch (Larix sibirica and Larix dahurica), growing likewise in the low places. The

larch is one of the most widely-spread trees of the "taiga". It requires less heat than the pine; in the region beyond the Baikal Mountains the larch is found on the northern slopes and the Scotch pine (Pinus silvestris) on the southern slopes. High up on the mountains climbs the Siberian pine (Pinus Cembra), which forms a timberline and a transition to the alpine meadows. Together with the poplar (Populus suaveolens) and Siberian pine, the larch reaches to the northern frontier of the spreading forest. In Kamchatka we meet with the larch (Larix dahurica), fir (Abies glacialis) and Erman birch (Betula ermani). To the southeast of the Tablan ridge the climate gets visibly softer and instead of conifers there appear hardwoods of a moderate line of the Asiatic continent, the elm (Ulmus campestris), daoura birch (Betula dahurica), Mongolian oak (Quercus mongolica), wild apple and nut. The climate of the Ussurian and Amurian countries, similar to that of Japan and China, renders possible the development of other species of conifers, and what is particularly noticeable, does so to a whole row of broad-leaved timbers. From among the conifers habitual to Siberian forests, the larch (Larix dahurica and Larix sibirica), spruce (Picea abovata) and silver fir (Pinus silvestris), is included the ajanian spruce (Picea ajanensis), Manchurian pine (Pinus koraiensis, Pinus funebris) and other species of fir (Abies Nephrolepis and Abies holophylla). Among the broad-leaved trees especially noticeable are the velvet or cork tree (Phellodendron amurense), white walnut (Dimorphanthus manchuria), Manchurian walnut (Juglans manchurica), two species of oak (Quercus mongolica and Q. grossorata), ash (Fraxinus manchurica), acacia (Cladrastris [Maackia] amurensis), 4 species of maple—out of these the most highly prized is the Acer tegmontosum and A. mono, 4 species of elm, 2 of basswood—Amurian (Tilia amurensis) and Manchurian (Tilia manchurica), 6 species of birch (the widest spread of them, Betula latifolia and Betula dahurica), poplar (Populus suaveolens) and aspen (Populus tremula).

The unlimited Siberian forests in their southern part gradually change into a forest-steppe zone. Even in the governments of Tobolsk and Tomsk, black soils are to be found, and in the place of conifers there occur more or less isolated

groves of aspen and birch. In the steppe regions of Acmolinsk and Semipalatinsk the birch and aspen give a tone to the forest. The poplar, black alder, acacia and conifers are met with less frequently.

On the mountains of Turkestan, especially on their northern slopes, a new species of spruce makes its appearance (Picea Schrenkiana), which sometimes grows together with the fir (Abies sibirica), the birch (Betula alba) and poplar (Populus laurifolia) at a height of 8000 to 10,000 feet above the level of the sea and exchanges place gradually with the juniper (Juniperus excelsa). In the central Turkestan mountains, especially where there is limestone, there appears a variegated vegetativeness of hardwood—walnut (Juglans regia), pistacio (Pistacia vera), white mulberry (Morus alba), oriental sycamore (Platanus orientalis), elm (Ulmus campestris), maple (Acer monospessulanum and Acer laetum), blackberry (Celtus australis) and others. In the river bottoms often grow the poplar (Populus euphratica), oleaster (Elaeagnus hortensis) and ash (Fraxinus potomorphia). The "saksaul" (Holoxylon amodendron) has, in many places, adapted itself to the dry continental climate and sandy soils of many of the semi-arid plains of Turkestan.

Growth of Trees.

The rate of growth of some of the species of forests in Asiatic Russia is shown by the following table:

Provinces	Species	Age Years	Height Feet	Diameter breast-high Inches
Government of Yeniseisk	Scotch pine (Pinus silvestris)	150	98	23
	Siberian pine (Pinus Cembra)	200	93	28
	Siberian larch (Larix sibirica)	200	103	23
Takeitsk Prov.	Siberian larch (Larix sibirica)	250	82	35
Amurian Prov.	Manchurian pine (Pinus koraiensis)	150	88	18
Sea-coast Prov.	Maritime fir (Abies holophylla)	150	75	13
	Mongolian oak (Quercus mongolica)	120	68	13
	Manchurian ash (Fraxinus manchurica)	120	80	16
	Elm (Ulmus campestris)	120	61	11
	Velvet (Phellodendron amurense)	100	51	12
	Walnut (Juglans manchurica)	100	68	12
	Acacia [Cladrastris (Maackia)]	100	50	11
	Amurian lime (Tilia amurensis)	120	75	15
	White birch (Betula latifolia)	100	75	14
	Black birch (Betula dahurica)	100	63	11
	Small-leaved maple (Acer mono.)	120	44	9

Mechanical Properties and Commercial Uses.

The quality of wood of the chief species of which the forests of Siberia are formed (the Scotch pine, Siberian pine, fir and spruce) are alike in their qualities to the similar wood of the northeast forests of European Russia.

Among the Siberian species, the larch and Scotch pine, as well as the European ones, are regarded as the most valuable materials for the erection of edifices, railroad bridges and ties.

The mechanical properties of the wood of the different species in the far east are recorded in the table on page 112.

The soft, light, easily-worked wood of the Manchurian pine is very like American white pine and is one of the best kinds of construction materials in the country.

The very dense hardwood of the Mongolian oak is good for the making of inlaid floors, railroad ties, wheel naves, hoops and veneering. The very valuable wood of the velvet, similar to the walnut, white and Manchurian walnut, is already used for veneering and in the future will have a still greater demand for finishing off cabinets, furniture and the best carpenter productions.

From among other hardwood species, the birch, elm and acacia are used for veneering, lime for fish-barrels, and aspen is exported to Japan for match-making.

Among the Turkestan timbers, the walnut is especially valued, the spunks on the trunks forming an object for exportation; the spruce, ash, elm, juniper and even the elaeagnus are regarded as building materials; the dense wood of the saksaul—its specific gravity exceeding 1 1/3 that of oak—is known for its heat-giving capacity, giving 30% more heat than the average European species.

Lumbering and Management.

The stumpage price for Scotch pine and Siberian larch varies from ½ to 5½ cents, the Siberian pine from ½ to 4 cents and the spruce and Siberian fir from ½ to 2 cents per cubic foot. In the seaside regions the stumpage saw-logs of Scotch pine, velvet, walnut and acacia are fixed at 1¼ to 2½ cents per cu. ft.; oak, ash, maple, elm and birch at ¾ to1½ per cubic foot. The Pacific Coast of the United States of America and China might import hardwood timber from seaside regions

Mechanical Properties of Timbers from Asiatic Russia.

SPECIES	Width of annual rings in millimetres	Percentage of moisture of wood sample	Specific gravity		Hardness of air-dry wood by method of Brinell and Tanka		Compressive strength parallel to grain		Modulus of Rupture
			Air dry	Absolute dry	Centre	Edge	Air dry	Absolute dry	
					Kilograms per sq. centimetre				
Manchurian pine (Pinus koraiensis)	1.5	11.5	0.440	0.419	300	260	482	691	88,728
Maritime fir (Abies holophylla)	2.4	11.9	0.371	0.341	280	260	360	510	68,847
Mongolian oak (Quercus mongolica)	2.2	13.4	0.781	0.748	840	930	554	1180	144,744
Manchurian ash (Fraxinus manchurica)	2.3	12.2	0.742	0.722	800	800	643	1251	144,247
Elm (Ulmus campestris)	2.2	12.0	0.515	0.493	510	400	421	743	75,278
Velvet (Phellodendron amurense)	2.9	10.2	0.523	0.502	500	430	538	841	103,794
Manchurian walnut (Juglans manchurica)	3.5	11.3	0.492	0.462	320	430	542	800	116,234
Amurian acacia [Cladrastris (Maackia) amurensis]	2.3	10.0	0.585	0.557	460	380	547	911	131,133
Manchurian lime (Tilia manchurica)	1.6	9.0	0.438	0.416	260	270	431	628	91,080
Black birch (Betula dahurica)	1.8	10.6	0.690	0.678	620	570	719	1231	143,935
White birch (Betula platyphylla)	1.8	10.5	0.648	0.630	490	530	662	1136	162,333
Small-leaved maple (Acer mono.)	0.7	9.4	0.710	0.682	860	890	630	1103	129,162

of Russia, where such valuable wood is obtainable at such a low price.

In the forests of Asiatic Russia the method of single-tree cutting predominates. With that system, very often it is not the largest trees that are cut down, but those more convenient to fell and transport, as used to be done in the French colonies and is continued to the present time in the Philippine Islands. In Siberia trees not more than 20 inches diameter at breast-height are cut down.

In the year 1912 it was decided to let go 1,035,254 thousand cubic feet of national wood; in reality there was sold and given for free use about 11% of the above, or 110,066 thousand cubic feet. In uninhabited regions trade in forest products is not developed and there is no demand for timber. Near villages fires are the chief misfortune of Siberian forests: the farmers, to do away with weeds and to better the pasture-ground, set fire to it, as is done in North America.

On reckoning up government forestry, it must be said that in all the national forests of the empire the output is about 3½ thousand million cubic feet of timber, out of which in reality not more than the half is sold and given for free use.

The wholesale income of the government in the course of 20 years has increased fourfold and at present reaches the sum of 45 million dollars, and the expenditure amounts to 14 million dollars.

ORGANIZATION AND PERSONNEL OF GOVERNMENT FOREST SERVICE.

In the government administration which forms part of the Department of Agriculture, there are about 4500 persons employed; out of this number 1500 are forest inspectors and supervisors and above 800 are forest assistants, examiners of mensuration for making of plans. These last receive a special forestry education in the Imperial Forest Institute, an institution with a 4-year course of study and about 700 students, and in the New Alexandria Institute of Agriculture, transferred for a time to Charkoff, with a special forest division, in which 80 to 120 students study. As aids to the foresters there are forest-rangers, who are prepared in 43 low forest schools for

two years, almost entirely at government expense. To guard the government forests there are 32,000 guards, about 1/3 of whom, in the capacity of horse-watchmen, are obliged to possess a horse.

CONSERVATION POLICY.

The private forests in Russia are under government control. According to the law of the year 1888, in private domains the protective and water-protected forests are dealt out. In private protective forests, as a preventive to the washing-out of soil, for protecting the appurtenances from sand-storms and so forth, the clearing out of soil is unconditionally forbidden and only the single-tree method of cutting is allowed. The government releases the protective forests from taxes, and taxes the organisation of their working plans upon its own account. In water-guarded forests which grow in places where the chief rivers take their source and have a special meaning for keeping up the quantity of water, the clearing out can only be admitted in such dimensions as will not have a bad effect on the sources of the rivers.

In all other private forests, according to the forest-protecting law, the owners must have working plans wherewith a desolate cutting is forbidden, as it exhausts the forest stand, a natural reproduction becomes impossible and the forest soil turns into barren land. The decision of the affairs concerning the conservation of private forests according to the law of 1888 is given into the charge of the forest-conservation committees, consisting of representatives of the highest government authorities, national forest-service personnel and private county delegates.

EXPORT OF TIMBER.

In the international market Russia occupies a priority. In the year 1912, Russia and Finland exported abroad timber valued at 124 million dollars, of which 40% was saw-logs, and other unmanufactured forms sold at a very low price. If we bear in mind that timber resources in many countries possessing forests have become much more scarce and that Russia has used no more than the half of the yearly growth of its forests, we must come to the conclusion that in the international tim-

ber market the importance of Russian timber will rise yearly and Russian forestry must expect a bright future.

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Paper No. 99

PRESERVATIVE TREATMENT OF TIMBER.

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Acknowledgments are due to our American railroads, treating plants, city engineers, and others, for their valuable assistance in furnishing much of the data upon which the durability tables given in this paper are based. Some of the data were also taken from an unpublished table compiled by C. P. Winslow. The Proceedings of the American Railway Engineering Association, and the American Wood Preservers' Association have been freely used.

PURPOSE OF THIS PAPER.

The purpose of this paper is to present a general review of the results obtained in the United States in preserving wood. The authors have attempted to do this by showing first, the quantity of wood preservatives used and amount of timber treated annually, and, second, the extent to which the various treatments have prolonged the natural life of wood. In addition, a partial bibliography is appended covering American practice.

It has not been possible to treat the subject in any detail in the brief space allotted the authors, hence, this paper should in no sense be considered as a treatise. The authors have made a laborious attempt to compile primarily for the use of engi-

neers, the best records they could find on the life of treated timbers. This information is given in Table 5. In view of the laxity with which treated timber has been handled and used, very few records contain information in sufficient detail, and because of this, their usefulness is greatly curtailed. The authors assume no responsibility whatever for the accuracy of any of the records given in the table except those for which the Forest Service is the authority. No cost data accompany the records, as these could not be obtained. There are doubtless many excellent records on the durability of treated timber that have not been given. The authors would deeply appreciate having additional records called to their attention.

STATUS OF INDUSTRY.

The wood preservation industry is aiding in the great movement for efficiency in operation and in the conservation of our National resources. Little known in our country a half-century ago, its growth, especially in the last decade, has been exceedingly rapid, until there are now nearly 100 plants in operation, producing over 150,000,000 cubic feet of treated timber annually. This rapid growth of the industry is shown in Figure 1 and Table 1, which give the number of plants in operation in the United States by years since 1860, and the amount of timber treated since 1908. Table 2 shows in detail the amount of each form of timber treated, since 1908, by different preservatives. Nearly 80 per cent of the total amount of timber treated is represented by cross-ties, and of these over 70 per cent were treated with creosote. The amount of each form of timber treated is shown graphically in Figure 2. The possibilities for further growth in this industry may be illustrated by the fact that in 1911 less than 22 per cent of the cross-ties used by the railroads were treated.

Of the 150,000,000 cubic feet of treated timber reported in 1913, about 70 per cent was treated with creosote, 24 per cent with zinc chloride, and most of the remainder with creosote and zinc chloride in combination. The amounts of these two preservatives used, by years, since 1908 are shown in Table 3 and in Figure 3. It is estimated that the value of these preservatives used in 1913 was between $9,000,000 and $10,000,000.

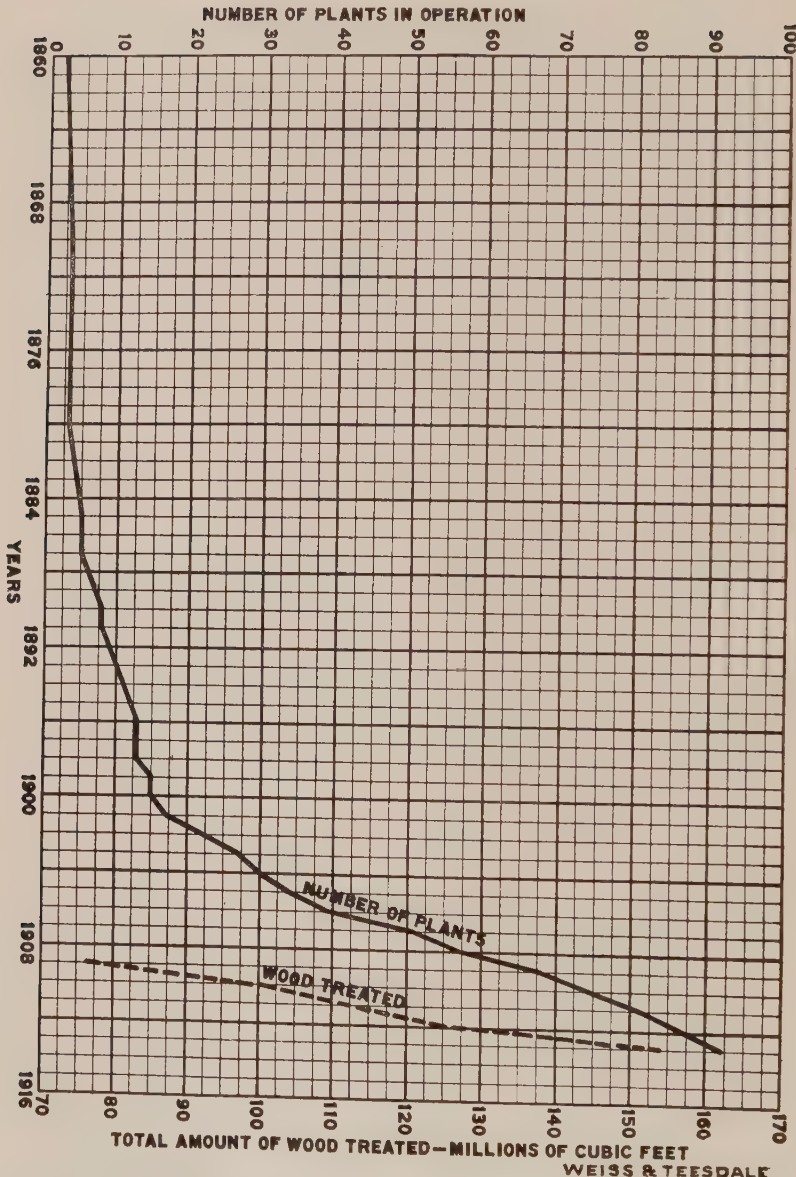


Fig. 1. The number of treating plants and the amount of wood treated in the United States.

PROCESSES PRACTICED IN PRESERVING WOOD.

Although a great many processes have been and are practiced in protecting timber from decay, they may logically be divided into two rather distinct groups, based upon the character of protection given. These may be termed the "superficial" and the "impregnation" processes. The superficial processes are those which aim to protect the wood by simply giving it a surface treatment. They include charring, seasoning, and brush and dipping treatments with preservatives.

By far the most important, both in the quantity of timber treated and in the degree of the protection obtained, are the impregnation processes. These may be conveniently divided into non-pressure processes, which include open tank treatments and kyanizing; and the pressure processes. In general, the best results are secured by the latter, in which the preservatives are pumped into the wood under pressure. The non-pressure processes, however, are well adapted to certain classes of work, such as the butt treatment of poles, and the local treatment of timber where it is impractical to have it sent to a pressure treating plant.

In Table 4 is given a list of the leading processes in general use in the United States, together with their patent numbers and names of the companies or individuals controlling them.

LIFE OF TREATED TIMBER IN THE UNITED STATES.

Definite data on the life of timber in service is of prime importance to the wood preserving industry, and especially to those interested in developing the various methods of treating timber. While great quantities of timber have been treated, practical data upon which reliable annual costs can be figured are incomplete. This is especially true of those patented processes which have come into extensive use within the last ten years. In early years, railroad companies and other consumers kept few records of their treated timber. Recently, many test tracks have been installed upon which very complete records are being kept, but these, in most cases, are still too recent to furnish durability data of much value.

In Table 5 are given the most reliable durability records that we have been able to obtain. As far as possible, only defi-

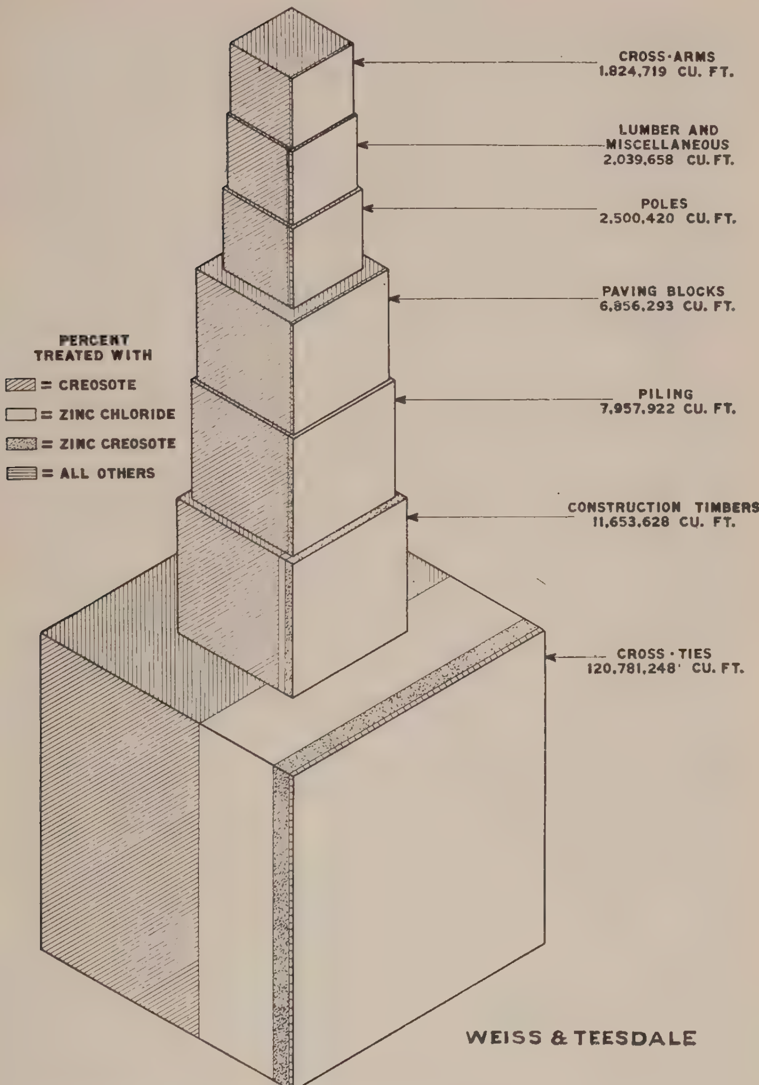
nite and accurate records on timber located in the United States have been included. A very large number of requests for durability records were sent to companies controlling patented processes and some proprietary preservatives, and also to railroad engineers and other consumers of treated timber. It is believed by the authors that the compilation from which the tables are taken is the most exhaustive ever prepared on this subject, and will be found of considerable practical value to those interested in the use of treated timber. In the case of some processes and preservatives, it is noted that no records are given. With some of the patented processes, the oldest records are less than ten years old, and in order to give some data on them the best records have been included. Good records, especially on ties treated by the Burnett, Wellhouse, and full cell processes, have been omitted in order to reduce the size of the table, and only the oldest and most reliable of these have been included. Furthermore, so far as possible, recent records on treated timber still in service have been eliminated. If the reader wishes a more comprehensive list of records than is included in this paper, the authors will be pleased to furnish it. Table 5 is classified by form of timber treated. An index is added, in which a classification for quick reference is made (1) by preservatives, and (2) by processes.

PREPARATION AND TREATMENT OF TIMBER.

If good results from the use of treated timber are to be obtained, it is of the greatest importance to have the timber properly prepared before the preservative is injected. Many failures can be traced to improper or insufficient preparation of the timber before the preservative is applied. The most important factors in the preparation of timber are: Peeling, seasoning, boring or framing, and the elimination of all decayed or partially decayed wood.

Largely because of its resistance to penetration, practically all preservative processes require the complete removal of bark before the wood can be best treated. Generally, this is best done immediately after the tree is felled. In sawed products, the bark is generally removed at the mill in slabbing the log.

As a rule, timber should be thoroughly seasoned before



THE AMOUNT OF TIMBER TREATED IN THE U.S. IN 1913

Fig. 2.

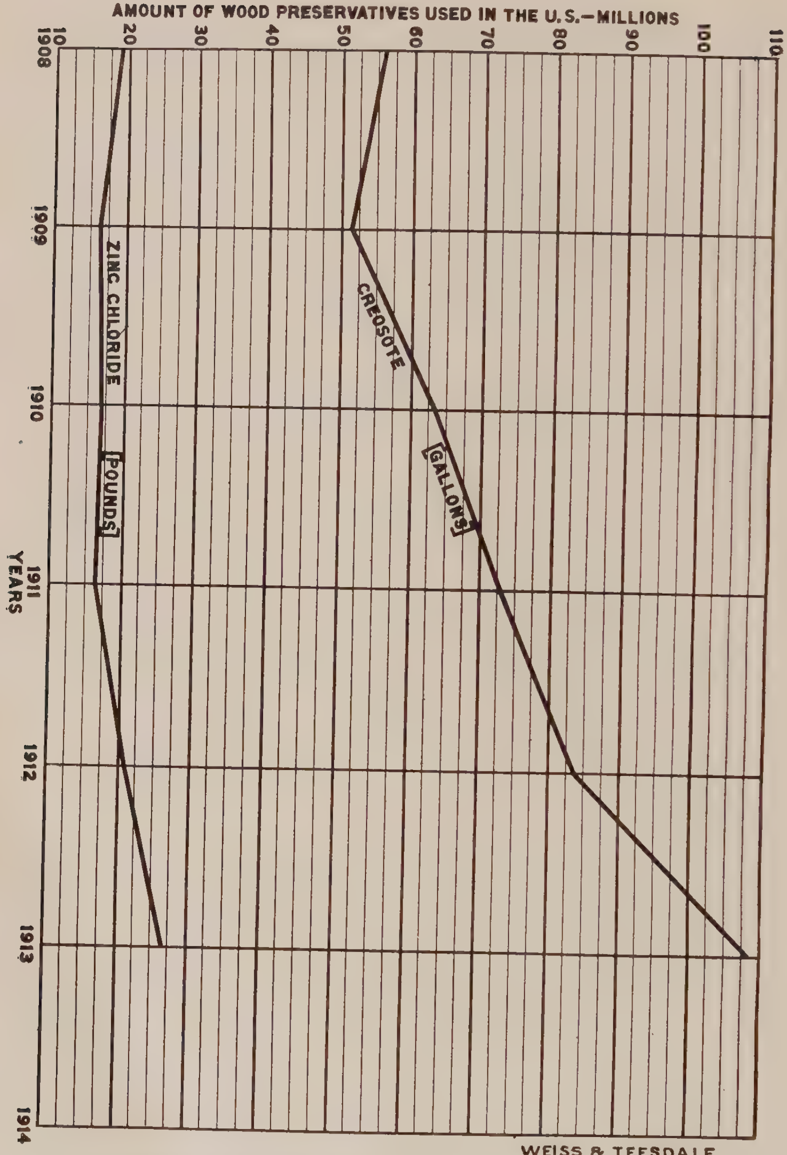


Fig. 3. The amount of creosote and zinc chloride used in the United States for preserving wood.

being treated. The wood in all living trees contains free water, and if this is not removed a satisfactory penetration with the preservative is generally impossible. Either air-seasoning or artificial seasoning is resorted to, though in general, air seasoning is preferred. In warm, damp climates or in cases where the timber must be treated soon after being felled, and also with certain species which do not treat well when air seasoned, artificial seasoning either by steaming or by boiling in the preservative is employed with good results.

Open air seasoning consists simply in piling the timber out of doors where it is exposed to the moisture. It should be piled in yards, preferably free from vegetation, decaying wood, and stagnant water, and the timbers so stacked that a free circulation of air can occur around each piece. Precautions should be taken by proper spacing to prevent checking and decay during the seasoning period. Seasoning in saturated steam is accomplished by running the timber into a retort and applying steam at about 20 pounds pressure for varying periods, dependent upon the amount of water it contains, its size, and the kind of wood. In the boiling process, the timber is heated in oil until the excess water is removed. It is the general consensus of opinion that in all of these artificial seasoning processes a temperature of 260° F. should not be exceeded. Lower temperatures than this are preferable, as there is liability to weaken the wood.

Of the various pressure processes in use, those given in Table 4 have the widest application in the United States. Of the treated cross-ties reported in 1913, 98 per cent were treated by the following six processes: Bethell (full cell), boiling, Burnett (zinc chloride), Lowry, Rueping, Card. Furthermore, most of the piling, paving blocks, and construction timbers were treated by these processes. They differ considerably in the method of injecting the preservative. In the Bethell, or full cell process, a preliminary vacuum is usually drawn and the preservative then applied under pressures varying from 100 to about 175 pounds per square inch. After the required amount has been injected, a final vacuum is sometimes drawn to remove the excess oil from the surface of the timber.

The Burnett process is carried out in practically the same manner as the Bethell, except that a solution of zinc chloride in water is used.

In the Card process, a mechanical mixture of creosote and zinc chloride is injected into the wood in much the same manner as in the above two methods.

The Lowry and Rueping processes are frequently called "empty cell" treatments, due to the fact that some of the oil injected into the timber is withdrawn. In the Lowry process no preliminary vacuum is applied, and the air that is confined in the timber before treatment is used with the aid of a quick high vacuum to force some of the oil out of the wood when the pressure is released. In the Rueping process, air is pumped into the timber before the preservative is applied. When the pressure is released, the expansion of this air forces a portion of the oil from the timber. A final vacuum is then applied to assist in the removal of excess oil from the wood. In the boiling process the timber is seasoned by boiling in the oil. This may then be followed by a full cell, or empty cell treatment.

CONCLUSIONS.

1. That the wood-preserving industry has become firmly established in the United States is shown by the fact that over 150,000,000 cubic feet of timber is being treated annually.

2. The increase in the amount of wood treated between 1908 and 1913 was 230 per cent.

3. This industry has made possible the use of immense quantities of non-durable timber, which, without treatment, would be but little used.

4. It has relieved the heavy demand upon the naturally durable woods, which were rapidly becoming exhausted.

5. Even when the more durable species are treated, their length of service is increased.

6. The industry has materially lessened the drain upon our forests, especially of the more valuable woods, due to the greater life given to timber subject to decay.

7. What engineers think of the practical value of wood preservation is shown by the growth in the number of plants and amount of wood treated.

8. The economic value of preserving wood is also shown by the records of durability, given in this paper, obtained from well-treated timber.

9. The extensive practice of preservative treatment has been too recent to make its influence on forest demands apparent. It seems likely, however, that, unless new uses are developed, the demand for treated timber will ultimately decrease because replacements will be made less frequently.

10. Six impregnation processes are now in general use in this country. These are the Bethell (full-cell creosote), boiling, Burnett (zinc chloride), Card, Lowry, and Rueping.

11. Considerable quantities of telephone poles, fence posts, and construction timbers are also treated by the open-tank, Kyanizing, dipping and brush methods.

12. The oldest records reported to the authors on the efficiency of preservative treatment are on the full-cell creosote, Burnett, and Wellhouse processes. For example, it is reported that 150,000 pine ties treated with 10 pounds of creosote per cubic foot gave 19 years average life on the H. and T. C. Railway; 12,000,000 Douglas fir ties treated with 0.27 pounds of zinc chloride per cubic foot gave from 10 to 12 years average life on the Southern Pacific Railway; 4,836,668 Douglas fir ties treated with 0.35 to 0.50 pounds of zinc chloride per cubic foot gave from 10 to 12 years average life on the C. B. & Q. Railway; 5,631,731 hemlock ties treated with 0.50 pounds of zinc chloride, by the Wellhouse process, gave 11 years average life on the C. R. I. & P. Railway.

13. Relatively little data are as yet available on the complete durability of timber treated by the Card, Lowry, or Rueping processes as applied in the United States.

14. Experience has shown that timber improperly prepared for treatment is very liable to give unsatisfactory service.

15. It is of prime importance to have timber properly peeled and seasoned before treatment, and also to be sure that it contains no advanced decay.

16. In general, the most approved method of seasoning is in the open air. Conditions sometimes demand artificial seasoning, which is satisfactorily accomplished either by steaming or boiling in oil.

17. High temperatures during artificial seasoning have been shown to injure the timber, and the best practice is not to exceed 260° F.

APPENDIX A.

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APPENDIX B.

TABLES.

Table 1. The Number of Wood Preserving Plants in the United States.

Year	Number of Plants	Year	Number of Plants	Year	Number of Plants
1860	2	1893	10	1904	30
1870	3	1894	11	1905	34
1880	3	1895	12	1906	39
1885	5	1896	13	1907	51
1886	5	1897	13	1908	57
1887	5	1898	13	1909	68
1888	6	1899	15	1910	75
1889	7	1900	15	1911	81
1890	8	1901	17	1912	87
1891	8	1902	22	1913	92
1892	9	1903	27		

Table 2. The Amount of Various Forms of Wood Treated with Different Preservatives in the United States in 1913.

Preservatives	Year	Cross-ties Cu. Ft.	Piling Cu. Ft.	Poles Cu. Ft.	Paving Blocks Cu. Ft.	Construction Timbers Cu. Ft.	Cross-arms Cu. Ft.	Lumber and Miscellaneous Cu. Ft.	Total material treated each year Cu. Ft.
Creosote	1908a	28,861,260	6,059,919	D	1,260,020	2,657,398	480,640	6,065,717	45,384,954
	1909b	29,830,080	4,421,726	659,664	2,994,290	4,902,311	41,764	417,787	43,267,622
	1910b	44,525,229	5,219,254	255,597	4,692,453	7,801,272	88,069	2,687,713	65,269,587
	1911b	49,532,163	4,937,363	106,213	10,145,724	7,417,105	71,961	2,499,995	74,710,524
	1912b	57,461,515	7,624,939	1,169,981	7,091,658	6,892,493	1,643,128	2,841,195	84,724,909
	1913b	75,998,307	7,630,328	2,367,769	6,810,308	10,308,883	1,813,010	1,853,993	106,782,598
Zinc Chloride	1908a	25,920,690	D	D	D	640,606	D	95,900	26,657,196
	1909b	24,153,162	D	D	D	320,891	D	2,333	24,476,386
	1910b	27,587,583	D	D	D	541,514	D	71,060	28,200,157
	1911b	28,337,883	D	D	D	1,043,851	D	119,931	29,501,665
	1912b	28,532,874	D	18,246	D	259,972	D	20,092	28,831,184
	1913b	36,051,816	D	47,996	D	585,756	D	7,670	36,693,238
Zinc-Creosote	1908a	9,781,590	426,610	D	D	95,700	D	35,858	10,339,758
	1909b	8,095,794	D	D	D	62,918	D	43,699	8,202,411
	1910b	6,354,219	38,392	D	D	181,143	D	30,646	6,604,400
	1911b	7,312,374	C	D	D	C	D	C	7,312,374
	1912b	8,214,303	97,874	D	D	560,613	D	99,367	8,972,157
	1913b	6,938,838	327,594	D	D	758,989	D	53,628	8,079,049
All Preservatives	1908a	64,563,540	6,486,529	D	1,260,020	3,393,704	480,640	6,197,475	82,381,908
	1909b	62,079,036	4,421,726	659,664	2,994,290	5,286,120	41,764	463,819	75,946,419
	1910b	78,467,031	5,257,646	255,597	4,692,453	8,523,929	88,069	2,789,419	100,074,144
	1911b	85,182,420	4,937,363	106,213	10,145,724	8,460,956	71,961	2,619,926	111,524,563
	1912b	97,183,009	7,737,035	1,188,579	7,397,095	7,793,524	1,643,128	2,988,686	125,931,056
	1913b	120,781,248	7,957,922	2,500,420	6,856,293	11,653,628	1,824,719	2,039,658	153,613,888

Note a—Figures furnished by the Wood Preservers' Association.
b—Figures furnished by United States Forest Service.
C—Figures, if used, would reveal identity of reporting firms.
D—No statistics.

Converting Factors.

To obtain the number of cross-ties, divide figures shown by 3.
To obtain the number of lineal feet of piling, divide the figures shown by .6763.
To obtain the number of lineal feet of poles, divide the figures shown by .5868.
To obtain the number of square yards of paving blocks, divide the figures shown by 2.625.
To obtain the number of board feet of construction timbers, divide the figures shown by .0833.
To obtain the number of cross-arms, divide the figures shown by .6198.
To obtain the number of board feet of lumber and miscellaneous material, divide the figures shown by .0833.

Table 3. Consumption of Wood Preservatives in the United States.

Year	Creosote Gallons	Zinc Chloride Pounds	Other Preservatives Gallons
1908	56,000,000	19,000,000	*
1909	51,431,212	16,215,107	*
1910	63,266,271	16,802,532	2,333,707**
1911	73,027,335	16,359,797	1,000,000**
1912	83,666,490	20,751,711	3,072,462**
1913	108,373,359	26,466,803	3,885,738**

*Figures not available.

**These figures represent the liquid preservatives in gallons for the respective years. Chemicals, like corrosive sublimate and aluminum sulphate, are reported in pounds. These preservatives are not listed, for the identity of firms would be revealed.

Table 4. The Principal Wood-Preserving Processes in use in the United States.

Name of Process	Preservative Used	Patent No.	Date patent was issued	Company or individual controlling patent
Bethell or full cell	Creosote	Expired		Open to all.
Boiling	Creosote	545,222	Aug. 27, 1895	
Burnett	Zinc Chloride	Expired		Open to all.
Card	Creosote and Zinc Chloride	815,404	Mar. 20, 1906	J. B. Card.
Kyanizing	Mercuric "	Expired		Open to all.
Lowry	Creosote	831,450	Sept. 18, 1906	American Creosote Co.
Open Tank	Creosote	Expired		Open to all.
Rueping	Creosote	709,799	Sept. 23, 1902	Lembcke von Bernuth Co.
Wellhouse	Zinc Chloride Glue and Tannin	Expired		Open to all.

INDEX TO TABLE 5. PART 1.—INDEXED BY PRESERVATIVE.

Preservative	Form	Process	Species	Record Number
Carbolineum-Avenarius	Mine Timbers	Open Tank	Beech	114
" "	" "	" "	Hemlock	115-116-117
" "	" "	" "	Maple	118
" "	" "	" "	Pine, Loblolly	119-120-121
" "	Poles	Brush	Cedar, W. Red	134
" "	"	"	" S. White	139
" "	"	"	Chestnut	137-138-140-141
" "	"	"	Pine, W. Yellow	135-136
Carbolineum-S. P. F.	"	"	Cedar, S. White	164-165
" "	"	"	Chestnut	162-163-166-167
Copper Sulphate	Ties	Thilmany	Pine, White	2
Creolin	Poles	Brush	Cedar, S. White	144
"	"	"	Chestnut	142-143
Creosote	Ties	"	Oak, White	26-27-28
"	"	Full Cell	Fir, Douglas	3
"	"	" "	Gum, Red	4
"	"	" "	Gum, Tupelo	5
"	"	" "	Oak, Red; Gum, Red	6
"	"	" "	Pine	7-8
"	"	Giussani	Oak, Red; Gum, Red	2-5
"	"	Lowry	....	9
"	"	Not Given	Cypress and Cedar	20
"	"	" "	Hemlock	21-22
"	"	" "	Pine	23
"	"	" "	Pine, Longleaf	24
"	"	Pressure	Oak, Red; Gum, Red	10
"	"	Rueping	Gum, Red	11
"	"	"	Oak, Red	12
"	"	"	Oak, Red; Gum, Red	13
"	"	"	Pine, Loblolly	14-15
"	"	"	Pine	16-17

"	"	"	Not Given	18-19
"	Piling	Boiling	Fir, Douglas	81-82
"	"	Full Cell	" "	83
"	"	" "	Oak, White	84
"	"	" "	Pine, S. Yellow	85 to 100 Incl.
"	"	" "	Pine, Sap	101
"	"	" "	Not Given	102 to 108 Incl.
"	Bridge Stringers	Boiling	Fir, Douglas	109
"	Bridge Timbers	Full Cell	Pine, Longleaf	110-111
"	" "	" "	" S. Yellow	112-113
"	Mine Timbers	" "	Fir, Red	122-123
"	" "	Open Tank	Hemlock	120-124-126-127
"	" "	" "	Pine, Loblolly	128-129-130
"	" "	" "	Pine	131-132
"	" "	" "	Pine, Loblolly and Shortleaf	133-133A-133B
"	" "	Full Cell	" " "	123A
"	Poles	Brush	Chestnut	150-152-153
"	"	"	Cedar, S. White	151-154
"	"	"	" W. Red	156
"	"	"	Pine, W. Yellow	155
"	"	Open Tank	Cedar, W. Red	148
"	"	" "	Chestnut	145-146
"	"	" "	Pine, W. Yellow	147-149
"	"	Full Cell	" S. Yellow	157-157A
"	Posts	Open Tank	Basswood	178
"	"	" "	Bay	179
"	"	" "	Cedar, White	180
"	"	" "	Cypress	181
"	"	" "	Gum, Sweet	182
"	"	" "	" Tupelo	183
"	"	" "	Oak, Red	184-185
"	"	" "	" Post	186
"	"	" "	Pine, Loblolly	187-188
"	"	" "	" S. Yellow	189
"	"	" "	" Oldfield	190
"	"	" "	Walnut, Black	191
"	"	" "	Willow	192

INDEX TO TABLE 5. PART 1.—INDEXED BY PRESERVATIVE.—Continued.

Preservative	Form	Process	Species	Record Number
Creosote	Paving Blocks	Dipping	Fir, Douglas	193-194
"	" "	Full Cell	Pine, Longleaf	195 to 199 Incl.
"	" "	" "	" Norway	203
"	" "	" "	Pine, Norway; Tamarack; Hemlock	200-201
"	" "	" "	Pine, Norway, and Longleaf; Fir, Douglas; Larch, West; Birch. White; Tamarack; Hemlock	202
"	" "	" "	Pine and Tamarack	204
"	Elec. Conduit	" "	Pine, Longleaf	233
"	----------------	Am. Creosote Wks.	----------------------------------	1
Creosote, Water-Gas Tar	Miscellaneous	Open Tank	Pine, S. Yellow	234
" " "	Mine Timbers	" "	Pine, Loblolly and Shortleaf	133C-133D
Creosote, Wood Tar	Poles	Brush	Chestnut	173-174
" " "	Ties	Dipping	Oak, White	30
" " "	"	"	" Red	29
Creosote and Resin	"	Creo-Resinate	Pine, S. Yellow	31
" "	Paving Blocks	"	" Longleaf	206-207-208-209-210-222
Crude Oil	Poles	Open Tank	Cedar, W. Red	159
" "	"	" "	Pine, W. Yellow	158
Imper. Wood Preserver	"	Brush	Cedar, S. White	161
" " "	"	"	Chestnut	160
Kreodone	Paving Blocks	Full Cell	Pine, Longleaf	211-212-213
"	" "	" "	Pine, Norway; Tamarack; Hemlock	214-215
Mercuric Chloride	Ties	Kyan	Hemlock	32
" "	"	"	Chestnut and Oak	33
Spirittine	Poles	Brush	Cedar, S. White	168
"	"	"	Chestnut	169
Tar Coating	"	"	"	170-171-172
Zinc Chloride	Ties	Burnett	Fir, Douglas	34 to 42 Incl.
" "	"	"	Gum, Red	43
" "	"	"	" Tupelo	44

" "	"	"	Hemlock	45-46-47
" "	"	"	Oak, Red	48-49
" "	"	"	Pine, Black Hills	50
" "	"	"	" Longleaf	51
" "	"	"	Pine	52
" "	"	"	Pine, Sap	53
" "	"	"	Pine and Fir, Douglas	54
" "	"	"	Tamarack	55-56
" "	"	"	Oak, Red; Elm; Ash; Beech	57
" "	Mine Timbers	"	Pine, Loblolly and Shortleaf	133G
" "	Poles	Open Tank	Cedar, W. Red	175
" "	"	" "	Pine, W. Yellow	176
Zinc Chloride and Creosote	....	Allardyce	....	57A
" " " "	Ties	Card	Not Given	64-64A
" " " "	"	Zinc-Creosote	Gum, Red	58
" " " "	"	"	" Tupelo	59
" " " "	"	"	Oak, Red	60
" " " "	"	"	Pine	61-62-66
" " " "	"	"	Pine, Sap	63
" " " "	"	"	Pine and Hemlock	65
" " " "	Poles	Open Tank	Cedar, W. Red	177
Zinc Tannin	Ties	Wellhouse	Gum	67
" "	"	"	Hemlock	68 to 73 Incl.
" "	"	"	Oak, Red	74
" "	"	"	Pine	75
" "	"	"	Pine, Sap	76
" "	"	"	" Shortleaf	77
" "	"	"	Tamarack	78-79
" "	"	"	Pine, White and Norway, and Tamarack	80
Not Stated	Paving Blocks	Not Stated	Cedar, Wash.	216
" "	" "	" "	Pine, Longleaf	217 to 221 Incl.
				223 to 229 Incl.
" "	" "	" "	Fir, Douglas	230
" "	" "	" "	Pine, Longleaf; Gum, Black	231
" "	" "	" "	Pine, Norway; Tamarack	232

INDEX TO TABLE 5. Part 2.—Indexed by Process.

Process	Form	Preservative	Species	Record Number
Am. Creosote Wks.	....	Creosote	....	1
Allardyce	....	Zinc Chloride and Creosote	....	57-A
Boiling	Piling	Creosote	Fir, Douglas	81-82
"	Bridge Stringers	"	" "	109
Burnett	Ties	Zinc Chloride	" "	34 to 42 Incl.
"	"	" "	Gum, Red	43
"	"	" "	" Tupelo	44
"	"	" "	Hemlock	45-46-47
"	"	" "	Oak, Red	48-49
"	"	" "	Pine, Black Hills	50
"	"	" "	" Longleaf	51
"	"	" "	Pine	52
"	"	" "	" Sap	53
"	"	" "	Pine and Fir, Douglas	54
"	"	" "	Tamarack	55-56
"	"	" "	Oak, Red; Elm; Ash; Beech	57
Brush	"	Creosote	Oak, White	26-27-28
"	Poles	"	Chestnut	150-152-153
"	"	"	Cedar, S. White	151-154
"	"	"	" W. Red	156
"	"	"	Pine, W. Yellow	155
"	"	Creosote, Wood Tar	Chestnut	173-174
"	"	Carbolineum-Avenarius	Cedar, W. Red	134
"	"	" "	" S. White	139
"	"	" "	Chestnut	137-138-140-141
"	"	" "	Pine, W. Yellow	135-136
"	"	Carbolineum-S. P. F.	Cedar, S. White	164-165
"	"	" "	Chestnut	162-163-166-167
"	"	Creolin	Cedar, S. White	144
"	"	"	Chestnut	142-143
"	"	Imper. Wood Preser.	Cedar, S. White	161

"	"	" " "	Chestnut	160
"	"	Spirittine	Cedar, S. White	168
"	"	"	Chestnut	169
"	"	Tar Coating	"	170-171-172
Card	Ties	Zinc Creosote	Not Given	64-64-A
Creo-Resinate	"	Creosote and Resin	Pine, S. Yellow	31
"	Paving Blocks	" " "	" Longleaf	206-207-208 209-210-222
Dipping	Ties	Creosote, Wood Tar	Oak, White	30
"	"	" " "	" Red	29
"	Paving Blocks	Creosote, Coal Tar	Fir, Douglas	193-194
Full Cell	Ties	" " "	" "	3
" "	"	" " "	Gum, Red	4
" "	"	" " "	" Tupelo	5
" "	"	" " "	Oak, Red; Gum, Red	6
" "	"	" " "	Pine	7-8
" "	Piling	" " "	Fir, Douglas	83
" "	"	" " "	Oak, White	84
" "	"	" " "	Pine, S. Yellow	85 to 100 Incl.
" "	"	" " "	" Sap	101
" "	"	" " "	Not Given	102 to 108 Incl.
" "	Bridge Timbers	" " "	Pine, Longleaf	110-111
" "	" "	" " "	" S. Yellow	112-113
" "	Mine Timbers	" " "	Fir, Red	122-123
" "	Paving Blocks	" " "	Pine, Longleaf	195 to 199 Incl.
" "	" "	" " "	" Norway	203
" "	" "	" " "	Tamarack	205
" "	" "	" " "	Pine, Norway; Tamarack; Hemlock	200-201
" "	" "	" " "	Pine, Norway and Longleaf; W. Larch; Fir, Douglas; Hemlock and Tamarack	202
" "	" "	" " "	Pine and Tamarack	204
" "	" "	Kreodone	Pine, Longleaf	211
" "	" "	"	Pine, Norway; Tamarack; Hemlock	214-215
" "	Elec. Conduit	Creosote	Pine, Longleaf	233
" "	Poles	"	" S. Yellow	157-157A
Giussani	Ties	"	Oak, Red; Gum, Red	25

INDEX TO TABLE 5. PART 2.—INDEXED BY PROCESS.—Continued.

Process	Form	Preservative	Species	Record Number
Kyan	Ties	Mercuric Chloride	Hemlock	32
"	"	" "	Chestnut and Oak	33
Lowry	"	Creosote	....	9
Not Given	"	"	Cypress and Cedar	20
" "	"	"	Hemlock	21-22
" "	"	"	Pine	23
" "	"	"	" Longleaf	24
Open Tank	Mine Timbers	"	Hemlock	120-124-126-127
" "	" "	"	Pine, Loblolly	128-129-130
" "	" "	"	Pine	131-132
" "	" "	"	Pine, Loblolly and Shortleaf	133
" "	" "	Carbolineum-Avenarius	Beech	114
" "	" "	" "	Hemlock	115-116-117
" "	" "	" "	Maple	118
" "	" "	" "	Pine, Loblolly	119-120-121
" "	" "	Zinc Chloride	" " and Shortleaf	133E-133F
" "	Poles	Creosote	Cedar, W. Red	148
" "	"	"	Chestnut	145-146
" "	"	"	Pine, W. Yellow	147-149
" "	"	Crude Oil	Cedar, W. Red	159
" "	"	" "	Pine, W. Yellow	158
" "	"	Zinc Chloride and Creosote	Cedar, W. Red	177
" "	"	Zinc Chloride	" "	175
" "	"	" "	Pine, W. Yellow	176
" "	Posts	Creosote	Basswood	178
" "	"	"	Bay	179
" "	"	"	Cedar, White	180
" "	"	"	Cypress	181
" "	"	"	Gum, Sweet	182
" "	"	"	" Tupelo	183
" "	"	"	Oak, Red	184-185

“ “	“	“	“ Post	186
“ “	“	“	Pine, Loblolly	187-188
“ “	“	“	“ S. Yellow	189
“ “	“	“	“ Oldfield	190
“ “	“	“	Walnut, Black	191
“ “	“	“	Willow	192
“ “	Miscellaneous	Creosote, Water Gas Tar	Pine, S. Yellow	234
Pressure	Ties	Creosote, Coal Tar	Oak, Red; Gum, Red	10
Rueping	“	“ “ “	Gum, Red	11
“	“	“ “ “	Oak, Red	12
“	“	“ “ “	“ “ and Gum, Red	13
“	“	“ “ “	Pine, Loblolly	14-15
“	“	“ “ “	Pine	16-17
“	“	“ “ “	Not Given	18-19
Thilmany	“	Copper Sulphate	Pine, White	2
Wellhouse	“	Zinc Tannin	Gum	67
“	“	“ “	Hemlock	68 to 73 Incl.
“	“	“ “	Oak, Red	74
“	“	“ “	Pine	75
“	“	“ “	“ Sap	76
“	“	“ “	“ Shortleaf	77
“	“	“ “	Tamarack	78-79
“	“	“ “	Pine, White and Norway; Tamarack	80
Zinc Creosote	“	Zinc Chloride and Creosote	Gum, Red	58
“ “	“	“ “ “ “	“ Tupelo	59
“ “	“	“ “ “ “	Oak, Red	60
“ “	“	“ “ “ “	Pine	61-62-66
“ “	“	“ “ “ “	“ Sap	63
“ “	“	“ “ “ “	Pine and Hemlock	65
Not Stated	Paving Blocks	Not Stated	Cedar, Wash.	216
“ “	“ “	“ “	Pine, Longleaf	217 to 221 Incl. and 223 to 229 Incl.
“ “	“ “	“ “	Fir, Douglas	230
“ “	“ “	“ “	Pine, Longleaf; Gum, Black	231
“ “	“ “	“ “	Pine, Norway; Tamarack	232

TABLE 5.

RECORDS ON THE LIFE OF TREATED TIMBER IN THE U. S. A.

Notation.

A = absorption of preservative in pounds per cu. ft.
B = location.
C = number originally placed.
D = year when placed.
E = length of service at last inspection, in years.
F = date of last inspection.
G = final condition of material at last inspection.
H = cause of removal.
I = remarks.
J = authority.

The numbers at the beginning of each paragraph are the record numbers, given in the last column of the preceding index. When reference letter is omitted, data are lacking.

Cross-Ties.

(1) Information requested but not furnished—Letter of Sept. 17, 1914, to American Creosote Works.

(2) **B**, N.Y. N.H. & H.Ry. — **C**, 1006 — **D**, 1881 — **E**, 15 — **F**, 1914 — **G**, all out in 1896 — **I**, average life, 14 years — **J**, A.R.E.A., Vol. 15, 1914.

(3) **A**, 8.6 — **B**, Maywood, Wash., N.P.Ry. — **C**, 442 — **D**, 1906 — **E**, 7.5 — **F**, 1914 — **G**, 7.7% removed — **H**, decay — **J**, Forest Service Report.

(4) **A**, 13.48 — **B**, Bay View & San Leon, Texas, G.H. & S.A.Ry. — **C**, 200 — **D**, 1905 — **E**, 9.0 — **F**, 1914 — **G**, 31% removed — **J**, Eng. Dept. G.H. & S.A.Ry.

(5) **A**, 12.3 — **B**, Bay View & San Leon, Texas, G.H. & S.A.Ry. — **C**, 201 — **D**, 1905 — **E**, 9.0 — **F**, 1914 — **G**, 26% removed — **J**, Eng. Dept. G.H. & S.A.Ry.

(6) **B**, Pacific, Mo., St.L. & S.F. — **C**, 76 — **D**, 1905 — **E**, 9.0 — **F**, 1914 — **G**, 1.3% removed — **H**, decay — **I**, 1 red gum tie removed — **J**, Report of St.L. & S.F.Ry., by V. K. Hendricks.

(7) **A**, 10.0 — **B**, Fair Haven, Conn., N.Y., N.H. & H.Ry. — **C**, 6000 — **D**, 1894 — **E**, 17 — **F**, 1914 — **G**, 99.6% removed — **I**, 20% were out by 1907 — **J**, W. J. Backes, Eng. M. of W., Letter of Dec. 10, 1914.

(8) **A**, 12.0 — **B**, Salix, La., G.H. & S.A.Ry. — **C**, 205 — **D**, 1907 — **E**, 7 — **F**, 1914 — **G**, none removed — **J**, Eng. Dept., G.H. & S.A.Ry.

(9) Average life to date was 10 years with no removals due to decay — No records furnished — Letter from Am. Creo. Co., Sept. 26, 1914.

(10) **B**, Pacific, Mo., St.L. & S.F.Ry. — **C**, 107 — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, 4% removed — **H**, decay — **J**, Report of St.L. & S.F. Ry., by V. K. Hendricks.

(11) **A**, 9.8 — **B**, St. Clair, Mo., St.L. & S.F.Ry. — **C**, 321 — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, 0.3% removed — **H**, split — **I**, 3% split (Mr. Lembcke does not consider these a fair test of Rueping process) — **J**, Report of St.L. & S.F.Ry., by V. K. Hendricks.

(12) **A**, 12.0 — **B**, St. Clair, Mo., St.L. & S.F.Ry. — **C**, 752 — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, 0.4% removed — **H**, decay — **I**, 2% show decay (Mr. Lembcke does not consider these a fair test of Rueping process) — **J**, Report of St.L. & S.F.Ry., by V. K. Hendricks.

(13) **B**, Pacific, Mo., St.L. & S.F.Ry. — **C**, 40 — **D**, 1905 — **E**, 9 — **F**, 1914 **G**, 7.5% removed — **H**, decay — **I**, 2 red gum and 1 red oak removed — **J**, Report of St.L. & S.F.Ry., by V. K. Hendricks.

(14) **A**, 5.0 — **B**, Corinth, Miss., I.C.Ry. — **C**, 3200 — **D**, 1907 — **E**, 7 — **F**, 1914 — **G**, one tie out 1914 — **I**, slightly rail cut — **J**, W. L. Parr, V. Pres.

(15) **A**, 5.0 — **B**, Corinth, Miss., I.C.Ry. — **C**, 2880 — **D**, 1907 — **E**, 7 — **F**, 1914 — **G**, one tie out in 1914 — **I**, slightly rail cut — **J**, W. L. Parr, V. Pres.

(16) **A**, not stated — **B**, Salix, La., G.H. & S.A.Ry. — **C**, 95 — **D**, 1907 — **E**, 7 — **F**, 1914 — **G**, none out — **J**, Eng. Dept., G.H. & S.A.Ry.

(17) **A**, not stated — **B**, Scott, La., G.H. & S.A.Ry. — **C**, 100 — **D**, 1907 — **E**, 7 — **F**, 1914 — **G**, none out — **J**, Eng. Dept., G.H. & S.A.Ry.

(18) **A**, 4.5 to 5.0 — **B**, A.T. & S.F.Ry. — **C**, 1,560,812 — **D**, 1906 — **E**, 8 — **F**, 1914 — **I**, no record of failures — **J**, Report by A.T. & S.F.Ry. to G. A. Lembcke.

(19) **A**, not given — **B**, A.T. & S.F.Ry. — **C**, 2,378,873 — **D**, 1907 — **E**, 7 — **F**, 1914 — **I**, no record of failures — **J**, Report by A.T. & S.F.Ry. to G. A. Lembcke.

(20) **B**, Central R.R. of N.J. — **C**, 500 — **D**, 1876 — **E**, 19 — **F**, 1895 — **G**, 100% out — **H**, rail cut — **I**, 10% removed in 1888 — **J**, J. O. Osgood, Ch. Eng. C.R. of N.J.

(21) **A**, 12.0 — **B**, Central R.R. of N.J. — **C**, 5000 — **D**, 1876 — **E**, 15.5 — **F**, 1908 — **H**, rail cut — **I**, average life 15.5 years — **J**, A.R.E.A., Vol. 10, p. 618.

(22) **B**, Medway, Mass., N.Y., N.H. & H.Ry. — **C**, 400 — **D**, 1880 — **E**, 32 — **F**, 1914 — **G**, all out in 1912 — **H**, none decayed, all rail cut — **I**, laid in a tunnel, average life 20 years — **J**, A.R.E.A., Vol. 13, p. 872; also letter Dec. 10, 1914, W. J. Backes.

(23) **A**, 10.0 — **B**, H. & T.C.Ry. — **C**, 150,000 — **D**, 1880 — **E**, 26 — **F**, 1906 — **G**, 92.5% removed — **I**, average life 19 years — **J**, A.R.E.A., Vol. 10, p. 619.

(24) **A**, no record — **B**, North Carolina, A.C.L.Ry. — **C**, 250 — **D**, 1887 — **E**, 23 — **F**, 1910 — **G**, 40% removed — **H**, rail cut, very few decayed — **I**, remainder in good condition — **J**, W. A. Fisher, Pres. of Am. Wood Preservers' Assn., 1911.

(25) **A**, no record — **B**, Missouri, St.L. & S.F.Ry. — **C**, 43 — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, 20.9% removed — **H**, decay — **J**, Report of St. L. & S.F.Ry., by V. K. Hendricks.

(26) **B**, Baltimore, Pa., lines — **C**, 597 — **D**, 1897-98 — **F**, 1914 — **H**, decay — **I**, 9 years average life — **J**, John Foley, Forester, Pa.Ry.

(27) **B**, Baltimore, Pa., lines — **C**, 541 — **D**, 1898 — **F**, 1914 — **H**, decay — **I**, 9.5 years average life — **J**, John Foley, Forester, Pa.Ry.

(28) **B**, Baltimore, Pa. lines — **C**, 386 — **D**, 1897 — **F**, 1914 — **H**, decay — **I**, 8.0 years average life — **J**, John Foley, Forester, Pa.Ry.

(29) **B**, Trenton, Pa. lines — **C**, 2910 — **D**, 1896 — **F**, 1914 — **H**, decay — **I**, 6.4 years average life — **J**, John Foley, Forester, Pa.Ry.

(30) **B**, N.Y. & Phil., Pa. lines — **C**, 348 — **D**, 1896-97 — **F**, 1914 — **H**, decay — **I**, 4.0 years average life — **J**, John Foley, Forester, Pa.Ry.

(31) **B**, Pa. lines east — **C**, 1250 — **D**, 1901 — **F**, 1914 — **H**, decayed and crushed — **I**, 9.0 years average life — **J**, John Foley, Forester, Pa. Ry.

(32) **B**, Mont. & Wyo., Bur. Mo. Ry. — **C**, 100,000 — **D**, 1884 — **F**, 1908 — **G**, splintered — **I**, average life 10 to 14 years — **J**, J. P. Snow.

(33) **B**, Baltimore, Nor. Cent. Ry. — **C**, 1 mile — **D**, 1838 — **E**, 11 — **F**, 1849 — **I**, sound after 11 years — **J**, O. Chanute, A.W.P.A., 1909.

(34) **A**, 0.25 — **B**, Southern Pacific Ry. — **C**, 300 miles of exper. track — **F**, 1914 — **H**, decay — **I**, 9.5 years average life — **J**, J. Q. Barlow, letter of Nov. 17, 1914.

(35) **A**, 0.35 to 0.50 — **B**, C.B.Q.Ry. — **C**, 4,836,668 — **D**, 1899 — **F**, 1914 — **H**, rot where covered by dirt — **I**, 9.5 years average life — **J**, A.R.E.A., Vol. 10, p. 619.

(36) **A**, 0.25 — **B**, Union Pacific Ry. — **C**, 300,000 per year since 1905 — **H**, 97% of removals due to decay — **I**, given 7 to 9 years average life — **J**, A. H. Mohler, Pres. U.P.Ry., letter of Dec. 7, 1914.

(37) **B**, Nevada, Oregon Short Line — **C**, 45 miles — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, 55% out — **H**, decay — **I**, 7 to 8 years average life — **J**, Eng. Dept., South. Pac. Ry.

(38) **B**, Utah, Ore. Short Line Ry. — **C**, 93 miles — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, 68% out — **H**, decay — **I**, 6 to 7 years average life — **J**, Eng. Dept., Southern Pac. Ry.

(39) **A**, 0.243 — **B**, Southern Pacific Ry. — **C**, 217 — **D**, 1895 — **F**, 1914 — **G**, all out — **H**, decay — **I**, 11.34 years average life — **J**, E. E. Calvin, V. Pres., S.P.R.R., letter of April 22, 1912.

(40) **B**, Southern Pacific Ry. — **C**, 4043 — **D**, 1896 — **F**, 1914 — **G**, all out — **H**, decay — **I**, 10.97 years average life — **J**, E. E. Calvin, V. Pres., S.P.R.R., letter of April 22, 1912.

(41) **B**, Southern Pacific Ry. — **C**, 1941 — **D**, 1897 — **F**, 1914 — **G**, all out — **H**, decay — **I**, 10.74 years average life — **J**, E. E. Calvin, V. Pres., S.P.R.R., letter of April 22, 1912.

(42) **B**, Southern Pacific Ry. — **C**, 20,536 — **D**, 1898 — **F**, 1914 — **G**, all out — **H**, decay — **I**, 11.21 years average life — **J**, E. E. Calvin, V. Pres., S.P.R.R., letter of April 22, 1912.

(43) **A**, 0.27 — **B**, Bay View & San Leon, Texas, G.H. & S.A.Ry. — **C**, 244 — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, 88% out — **J**, Eng. Dept., G.H. & S.A.Ry.

(44) **A**, 0.23 — **B**, Bay View & San Leon, Texas, G.H. & S.A.Ry. — **C**, 198 — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, 90% out — **J**, Eng. Dept., G.H. & S.A.Ry.

(45) **B**, Pa. lines west — **C**, 11,393 — **D**, 1896-97 — **E**, 8 — **F**, 1905 — **G**, 22% out — **J**, A.R.E.A., Vol. 8, p. 469.

(46) **B**, Pa. lines west — **C**, 713 — **D**, 1898 — **E**, 7 — **F**, 1905 — **G**, 8% out — **J**, A.R.E.A., Vol. 8, p. 469.

(47) **B**, Chicago, C.R.I. & P.Ry. — **C**, 2000 — **D**, 1866 — **E**, 15 — **F**, 1881 — **G**, 25% out — **J**, Trans. Am. Soc. Civil Eng., 1899, p. 349.

(48) **B**, Pittsburg, Pa. lines — **C**, 135 — **D**, 1897 — **E**, 8 — **F**, 1905 — **G**, 26% out — **J**, A.R.E.A., Vol. 8, p. 469.

(49) **A**, 0.33 — **B**, S. Dakota, C.B.Q.Ry. — **C**, 550 — **D**, 1900 — **E**, 14 — **F**, 1914 — **G**, 13% out — **H**, decay — **I**, 100% gave 12 years life — **J**, J. H. Waterman.

(50) **A**, 0.33 to 0.4 — **B**, S. Dakota, C.B.Q.Ry. — **C**, 6354 — **D**, 1900 — **E**, 14 — **F**, 1914 — **G**, 6% out — **H**, decay — **J**, J. H. Waterman.

(51) **B**, Rowlyton, Conn., N.Y., N.H. & H.Ry. — **C**, 206 — **D**, 1901 — **E**, 13 — **F**, 1914 — **G**, 27% out — **H**, rail cut — **I**, none removed until 1914 — **J**, A.R.E.A., Com. on Wood Preser.

(52) **A**, 0.25 — **B**, Texas & New Orleans Ry. — **C**, 10,500,000 — **D**, 1888 — **F**, 1908 — **H**, decay — **I**, 10 to 11 years average life — **J**, A.R.E.A., Vol. 8, p. 469.

(53) **A**, 0.41 — **B**, Bay View & San Leon, Texas, G.H. & S.A.Ry. — **C**, 1001 — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, 98.3% out — **J**, Eng. Dept. G.H. & S.A.Ry.

(54) **A**, 0.27 — **B**, South. Pacific Ry. — **C**, 12,000,000 — **D**, 1895 — **F**, 1908 — **H**, checks and dry rot — **I**, 10 to 12 years average life — **J**, A.R.E.A., Vol. 10, p. 619.

(55) **B**, Wyoming, C.B.Q.Ry. — **C**, 1320 — **E**, 10.4 — **G**, 60.7% removed — **H**, decay — **J**, J. H. Waterman.

(56) **A**, 0.31 to 0.51 — **B**, Janesville, Wis., C. & N.W.Ry. — **C**, 388 — **D**, 1907 — **E**, 6.5 — **F**, 1914 — **G**, no removals — **I**, 96.6% in good condition — **J**, Forest Service.

(57) **A**, 0.50 — **B**, Tenn., I.C.Ry. — **C**, 3080 — **D**, 1908 — **E**, 6.0 — **F**, 1914 — **G**, 6% removed — **H**, decay — **I**, 20% affected by decay — **J**, W. L. Parr, V. Pres.

(57-A) Information requested but not furnished — Letters of September 15 and Oct. 3, 1914 — **J**, Internat. Creo. Const. Co.

(58) **B**, Bay View, Tex., G.H. & S.A.Ry. — **C**, 45 — **D**, 1905 — **E**, 9.0 — **F**, 1914 — **G**, 33% removed — **J**, Eng. Dept., G.H. & S.A.Ry.

(59) **B**, Bay View, Tex., G.H. & S.A.Ry. — **C**, 42 — **D**, 1905 — **E**, 9.0 — **F**, 1914 — **G**, 69% removed — **J**, Eng. Dept., G.H. & S.A.Ry.

(60) **B**, Scott, La., G.H. & S.A.Ry. — **C**, 95 — **D**, 1907 — **E**, 7.0 — **F**, 1914 — **G**, 1.6% out — **J**, Eng. Dept., G.H. & S.A.Ry.

(61) **A**, 5-6 lbs. Creosote — **B**, Scott, La., G.H. & S.A.Ry. — **C**, 547 — **D**, 1907 — **E**, 7.0 — **F**, 1914 — **G**, 2.7% out — **J**, Eng. Dept., G.H. & S.A.Ry.

(62) **A**, not stated — **B**, Watkins cut off, G.H. & S.A.Ry. — **C**, 1694 — **D**, 1895 — **E**, 19 — **F**, 1914 — **G**, 92.6% out — **J**, Eng. Dept., G.H. & S.A.Ry.

(63) **A**, Zn. 0.27, Cr. 6.8 — **B**, Bay View, Texas, G.H. & S.A.Ry. — **C**, 1003 — **D**, 1905 — **E**, 9.0 — **F**, 1914 — **G**, 20.8% removed — **J**, Eng. Dept., G.H. & S.A.Ry.

(64) **A**, not given — **B**, C.B. & Q.Ry. — **C**, 15,819 — **D**, 1909-10 — **E**, 4 and 5 — **F**, 1914 — **G**, 1.5% removed — **H**, all causes — **I**, 0.25% for decay, 0.75% split and broken, 0.5% miscellan. — **J**, J. H. Waterman, letter of Feb. 24, 1915.

(64-A) **A**, Zn. 0.5 lb., Cr. 2 lbs. — **B**, C.M. & St.P.Ry., Mo. — **C**, 596 — **D**, 1908 — **E**, 6.0 — **F**, 1914 — **G**, none out — **I**, in good condition — **J**, F. S. Pooler, letter Feb. 23, 1915.

(65) **A**, 5-6 lbs. Creosote — **B**, Salix, La., G.H. & S.A.Ry. — **C**, 433 — **D**, 1907 — **E**, 7.0 — **F**, 1914 — **G**, 4.4% removed — **J**, Eng. Dept., G.H. & S.A.Ry.

(66) **B**, Ft. Hancock, G.H. & S.A.Ry. — **C**, 200 — **D**, 1894 — **E**, 19 — **F**, 1914 — **G**, 64% removed — **H**, dry rot — **J**, Eng. Dept., G.H. & S. A.Ry.

(67) **B**, Norfolk Div., N. & S. — **C**, 250 — **D**, 1897 — **E**, 17 — **F**, 1914 — **G**, 69% out — **J**, Report, N. & S.Ry., 1914.

(68) **B**, S.C. & S.R.Ry. — **C**, 1287 — **D**, 1897 — **E**, 8 — **F**, 1905 — **G**, 30% out — **J**, A.R.E.A., Vol. 8, p. 469.

(69) **A**, 0.25 to 0.50 — **B**, C.R.I. & P.Ry. — **C**, 5,031,731 — **D**, 1886 — **F**, 1908 — **H**, decay and rail cut — **I**, average life, 10 years east and 11.7 years west of Mo. River — **J**, A.R.E.A., Vol. 10, p. 618.

(70) **A**, 0.30 — **B**, Union Pacific Ry. — **C**, 150,000 — **D**, 1886 — **F**, 1908 — **G**, rail worn — **I**, 12 to 15 years average life — **J**, A.R.E.A., Vol. 10, p. 618.

(71) **B**, Hanna, Penn. lines — **C**, 200 — **D**, 1892 — **E**, 13 — **F**, 1905 — **G**, all out — **I**, 10.7 years average life — **J**, Railway Gazette, 1905, Vol. 38, p. 282.

(72) **B**, C. & N.W.Ry. — **C**, 1534 — **D**, 1907 — **E**, 6.5 — **F**, 1914 — **G**, none removed — **I**, 94.6% in good condition — **J**, Forest Service.

(73) **A**, 0.50 — **B**, C.R.I. & P.Ry. — **C**, 21,850 — **D**, 1886 — **E**, 12 — **F**, 1898 — **G**, 63.1% out — **I**, 11.8 years average life — **J**, A.S.C.E., Vol. 42, p. 303.

(74) **B**, Joppa, Ill., C. & E.I.Ry. — **C**, 24,271 — **D**, 1900 — **E**, 14 — **F**, 1914 — **G**, 25.2% out — **H**, decay — **J**, J. H. Waterman, Proc. A.W.P.A., 1915.

(75) **A**, 2.7#/ tie — **B**, Norfolk Div., N. & S. — **C**, 250 — **D**, 1897 — **E**, 17 — **F**, 1914 — **G**, 66% out — **J**, Report, N. & S.R.R., 1914.

(76) **B**, M.K. & T.Ry. — **C**, 1,300,612 — **D**, 1901 — **E**, 6.0 — **F**, 1907 — **G**, 22% out — **J**, A.R.E.A., Vol. 10, p. 612.

(77) **B**, Wash. Iowa, C.M. & S.P.Ry. — **C**, 500 — **D**, 1902 — **E**, 12 — **F**, 1914 — **G**, 10% out — **H**, decay — **J**, F. S. Pooler, letter Sept. 23, 1914.

(78) **A**, .41-.44 — **B**, Janesville, Wis., C. & N.W.Ry. — **C**, 738 — **D**, 1907 — **E**, 6.5 — **F**, 1914 — **G**, no removals — **I**, 96.3% in good condition — **J**, Forest Service.

(79) **B**, Koscinsko, Ind., Pa. lines — **C**, 200 — **D**, 1892 — **E**, 13 — **F**, 1905 — **G**, all out — **J**, Railway Gaz., 1905, Vol. 38, p. 282.

(80) **B**, D. & I.R.Ry. — **C**, 256 — **D**, 1890 — **E**, 18 — **F**, 1914 — **G**, nearly all out in 1911 — **H**, part removed for decay — **I**, estimated life 15 years — **J**, Letter to Mr. Angier by W. H. Clark, Eng.

Piling.

(81) **A**, 10.0 — **B**, California — **C**, 16,000 driven since 1903 — **E**, 17 years average — **F**, 1914 — **I**, 269 showed decay in center, with shell sound; not subject to marine borers — **J**, J. Q. Barlow, Asst. Chief Eng., Southern Pacific Rwy.

(82) **B**, Arizona-New Mexico — **C**, 9000 — **I**, 539 showed decay in center, with shell sound; not subject to marine borers — **J**, J. Q. Barlow, Asst. Chief Eng., Southern Pacific Rwy.

(83) **B**, Washington — **C**, bridge 3 miles long — **D**, 1891 — **E**, 22 — **F**, 1913 — **G**, failure from teredo negligible — **H**, superseded by fill in 1913 — **I**, some removals because of decay where cut for drift bolts — **J**, G. A. Colman, of Colman Creosote Works.

(84) **A**, 8.0 — **B**, Sandy Hook, N.Y. — **C**, 97 — **D**, 1903-4 — **E**, 8.5 — **F**, 1912 — **G**, good — **I**, ship worms active in this locality in warm weather — **J**, U. S. War Dept.

(85) **A**, 14.0 — **B**, Sandy Hook, N. Y. — **C**, 448 — **D**, 1903-4 — **E**, 8.5 — **F**, 1912 — **G**, good — **I**, ship worms active in this locality in warm weather — **J**, U. S. War Dept.

(86) **A**, 20.0 — **B**, Navy Yard, Pensacola, Florida — **C**, 198 — **D**, 1902 — **E**, 3.5 — **F**, 1906 — **G**, 6 attacked, 2 destroyed — **H**, ship worms attacked — **I**, attacked in defective sections where oil did not penetrate well — **J**, H. R. Stanford, Trans. Am. Soc. C. E., 1906.

(87) **A**, 20-24 — **B**, Galveston, Texas — **C**, 150 — **D**, 1875 — **E**, 28 — **F**, 1903 — **G**, comparatively intact — **J**, "A Treatise on Wood Preservation", Inter. Creo. & Const. Co.

(88) **A**, 24 — **B**, Galveston, Texas — **C**, 3000 — **D**, 1895 — **E**, 18 — **F**, 1913 — **G**, 90% still good — **I**, oil contained 60% naphthalene and 12-15% tar acids — **J**, F. B. Ridgway, Proc. Am. Wood Pres. Assn., 1914.

(89) **A**, 15-16 — **B**, L. & N.R.R., N.O. & N.Div., in salt water — **C**, 5100 — **D**, 1877-78 — **E**, 30-31 — **F**, 1908 — **G**, see remarks — **H**, teredo attack and change in construction — **I**, 81% in service but protected with cement and pipe, 19% cut off or replaced — **J**, J. B. Lindsey, L. & N.Ry.

(90) **B**, Pensacola, Fla. — **C**, about 1000 — **D**, 1898 — **E**, 16 — **F**, 1915 — **G**, all show some attack, some being replaced — **H**, attacked by xylotrya, limnoria and pholas — **I**, located at Commendencia Street wharf — **J**, J. B. Lindsey, L. & N.Ry.

(91) **B**, Pensacola, Fla. — **C**, about 100 — **D**, 1895 — **E**, 19 — **F**, 1915 — **G**, all bad shape, part removed — **H**, attacked by xylotrya, limnoria and pholas — **I**, bulkhead piles located at Terregonna St. wharf — **J**, J. B. Lindsey, L. & N.Ry.

(92) **B**, Pensacola, Fla. — **C**, about 300 — **D**, 1901 — **E**, 13 — **F**, 1915 — **G**, all badly attacked — **H**, attacked by xylotrya, limnoria and pholas — **I**, bulkhead piles located at Jefferson Street wharf — **J**, J. B. Lindsey, L. & N.Ry.

(93) **B**, Pensacola, Fla. — **D**, 1881 — **E**, 23 — **F**, 1915 — **G**, all badly attacked — **H**, attacked by xylotrya, limnoria and pholas — **I**, Muscogee wharf,—these piles are still in place but do not support any load,—all are attacked and many destroyed — **J**, J. B. Lindsey, L. & N.Ry.

(94) **A**, 18 — **B**, Gulfport, Miss. — **C**, about 200 — **D**, 1902 — **E**, 7 — **F**, 1909 — **G**, all replaced — **H**, teredo attack — **J**, G. & S.I.Ry. and Forest Service.

(95) **A**, 18-20 — **B**, Gulfport, Miss. — **C**, 196 — **D**, 1909 — **E**, 5 — **F**, 1914 — **G**, 11.8% destroyed, 39.0% sound — **H**, teredo attack — **I**, the remaining piles were slightly and mediumly attacked — **J**, G. & S.I.Ry. and Forest Service.

(96) **A**, 8.5 — **B**, Gulfport, Miss. — **C**, 4 — **D**, 1912 — **E**, 2 — **F**, 1914 — **G**, all slightly attacked — **I**, F.S. Exper. piles subject to attack by ship worm — **J**, G. & S.I.Ry. and Forest Service.

(97) **A**, 10.7 — **B**, Pensacola, Fla. — **C**, 3 — **D**, 1913 — **E**, 1 — **F**, 1914 — **G**, 2 slightly attacked, 1 mediumly attacked — **I**, F.S. Exper. piles subject to attack by ship worm, limnoria and pholas — **J**, G. & S.I. Ry. and Forest Service.

(98) **A**, 20.7 — **B**, Pensacola, Fla. — **C**, 4 — **D**, 1913 — **E**, 1 — **F**, 1914 — **G**, 2 slightly attacked, 1 mediumly attacked — **I**, F.S. Exper. piles subject to attack by ship worm, limnoria and pholas — **J**, G. & S.I. Ry. and Forest Service.

(99) **A**, 7.6 — **B**, San Diego, Cal. — **C**, 4 — **D**, 1912 — **E**, 2 — **F**, 1914 — **G**, 2 slightly attacked, 1 mediumly attacked — **I**, F.S. Exper. piles subject to attack by ship worm and limnoria — **J**, G. & S.I.Ry. and Forest Service.

(100) **A**, 20.8 — **B**, Fort Mason, Cal. — **C**, 4 — **D**, 1911 — **E**, 3 — **F**, 1914 — **G**, all slightly attacked — **I**, F.S. Exper. piles subject to attack by ship worm and limnoria — **J**, G. & S.I.Ry. and Forest Service.

(101) **A**, 25 — **B**, Manatee River, Fla., Seaboard Airline Ry. — **C**, bridge — **D**, 1900 — **E**, 14 — **F**, 1914 — **G**, perfect — **I**, oil contained 50% naphthalene — **J**, E. S. Christian, Proc. Am. Wood Preser. Assn., 1915.

(102) **A**, 12 — **B**, Charleston, S. C., S.R.R., Pier 1 — **D**, 1911-12 — **E**, 2-3 — **F**, 1914 — **G**, see remarks — **I**, some very bad piles, majority in good condition — **J**, T. G. Townsend, Southern Ry.

(102-A) **A**, 12 — **B**, Charleston, S. C., S.R.R., Pier 2 — **D**, 1912 — **E**, 2 — **F**, 1914 — **G**, see remarks — **I**, practically every pile attacked by limnoria, many destroyed — **J**, T. G. Townsend, Southern Ry.

(103) **A**, 12 — **B**, Charleston, S. C., S.R.R., Pier 3 — **D**, 1913 — **E**, 1 — **F**, 1914 — **G**, see remarks — **I**, practically every pile attacked by limnoria, many destroyed — **J**, T. G. Townsend, Southern Ry.

(104) **A**, 16-20 — **B**, Norfolk Navy Yard, Pier No. 311 — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, see remarks — **H**, attacked by teredo — **I**, every pile attacked by teredo, most of them worthless, now being replaced — **J**, T. G. Townsend, Southern Ry.

(105) **A**, 20 — **B**, Norfolk Navy Yard, Slip No. 318 — **D**, 1908 — **E**, 6 — **F**, 1914 — **G**, good — **I**, slightly attacked — **J**, T. G. Townsend, Southern Ry.

(106) **A**, 18 — **B**, Charleston, S. C., wharf of Oakdine Compr. Co. — **D**, 1912 — **E**, 2 — **F**, 1914 — **G**, no removals — **H**, limnoria attack — **I**, some of the piles have 20% of cross-section gone — **J**, T. G. Townsend, Southern Ry.

(107) **A**, 20 — **B**, Charleston, S. C., wharf of S. C. Warehouse Corp. — **C**, 400 — **D**, 1909 — **E**, 5 — **F**, 1914 — **G**, no removals — **H**, limnoria attack — **I**, piles show some recent attack — **J**, T. G. Townsend, Southern Ry.

(108) **A**, 12 — **B**, Newport News, C. & O. Pier No. 6 — **C**, 4000 — **D**, 1883 — **E**, 31 — **F**, 1914 — **G**, in use — **I**, oil contained 60% naphthalene — **J**, E. S. Christian, Proc. Am. Wood Preser. Assn., 1915.

Bridge Timbers.

(109) **A**, 10 — **B**, Southern Pacific Ry. — **C**, 15,000,000 — **D**, 1896-00 — **E**, 14-18 — **F**, 1914 — **G**, good — **I**, bridge stringers — **J**, J. Q. Barlow, Asst. Ch. Engr., S.P.Ry.

(110) **A**, 12 — **B**, Delaware, Lackawanna and Western Ry. — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 100% removed — **H**, see remarks — **I**, bridge ties removed because of shattered condition. Supposed to be due to a preliminary steaming at 25 lbs. for 8.25 hours before treating — **J**, Eng. Dept., D.L. & W.Ry.

(111) **A**, 10 — **B**, Brooklyn Bridge, N. Y. — **D**, 1882 — **E**, 27 — **F**, 1909 — **G**, good — **I**, a number removed during track changes were sound and good — **J**, L. B. Shipley, Barrett Mfg. Co.

(112) **B**, L. & N.Ry., N.O. & M.Div. — **C**, 4106 — **D**, 1876-8 — **F**, 1897 — **G**, all removed — **I**, all the timbers under record No. 112 were bridge ties. About 30% were lost in a storm in 1893. Estimated that 45 to 55% of the total number were still in service in 1896 — 12% in 1903 and 0.41% in 1905 — **J**, John B. Lindsey, Supt. West Pascagoula Creosoting Works, letter Oct. 14, 1914.

(112) **B**, L. & N.Ry., N.O. & M.Div. — **C**, 400 — **D**, 1876 — **F**, 1903 — **G**, all removed — **I**, see above — **J**, see above.

(112) **B**, L. & N.Ry., N.O. & M.Div., Bay St. Louis, Miss. — **C**, 5000 — **D**, 1878-79 — **E**, 25 — **F**, 1904 — **G**, all removed — **I**, see above — **J**, see above.

(112) **B**, L. & N.Ry., N.O. & M.Div., Bay St. Louis, Miss. — **C**, 1560 — **D**, 1878-79 — **E**, 25 — **F**, 1905 — **G**, all removed; 1166 removed by 1903 — **I**, see above — **J**, see above.

(112) **B**, L. & N.Ry., N.O. & M.Div., Biloxi Bay, Miss. — **C**, 1920 — **D**, 1878-79 — **E**, 30 — **F**, 1905 — **G**, 76 in service 1905 — **I**, see above — **J**, see above.

(112) **B**, L. & N.Ry., N.O. & M.Div., Biloxi Bay, Miss. — **C**, 1893 — **D**, 1878-79 — **E**, 30 — **F**, 1905 — **G**, 533 in service 1905 — **I**, see above — **J**, see above.

(113) **A**, 16 — **B**, New Orleans, La. — **C**, several wharves — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, good — **I**, some decay in heartwood where tops cut off. Placed in fresh water — **J**, Samuel Young, Board of Commission of the Port of New Orleans.

Mine Timbers.

(114) **A**, 1.1 — **B**, Scranton, Pa., Dodge Coll. Drift — **C**, 1 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, slightly decayed — **J**, Forest Service.

(115) **A**, 0.73 — **B**, Scranton, Pa., Dodge Coll. Drift — **C**, 2 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, all slightly decayed — **J**, Forest Service.

(116) **B**, Scranton, Pa., Bellevue Mine — **C**, 3 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, all sound — **J**, Forest Service.

(117) **B**, Scranton, Pa., Bellevue Mine — **C**, 2 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, all sound — **J**, Forest Service.

(118) **A**, 1.0 — **B**, Scranton, Pa., Dodge Coll. Drift — **C**, 1 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, all sound — **J**, Forest Service.

(119) **A**, 1.7 — **B**, Scranton, Pa., Dodge Coll. Drift — **C**, 10 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, 30% slt. decayed, 10% split, 60% sound — **J**, Forest Service.

(120) **B**, Scranton, Pa., Bellevue Mine — **C**, 10 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, 70% sound — **I**, 30% removed but not for decay — **J**, Forest Service.

(121) **B**, Scranton, Pa., Bellevue Mine — **C**, 2 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, all sound — **J**, Forest Service.

(122) **A**, 1.5 — **B**, Bunker Hill & Sullivan Mining & Concentrating Co., Kellogg, Idaho — **C**, 27 — **D**, 1908 — **E**, 3 — **F**, 1911 — **G**, good — **J**, Forest Service.

(123) **A**, 2.72 — **B**, Bunker Hill & Sullivan Mining & Concentrating Co., Kellogg, Idaho — **C**, 93 — **D**, 1908 — **E**, 3 — **F**, 1911 — **G**, good — **J**, Forest Service.

(123-A) **A**, 10 — **B**, Pottsville, Pa., P. and R. Mines — **C**, 87 — **D**, 1907 — **E**, 3 — **F**, 1910 — **G**, 76% sound — **H**, crush and squeeze — **I**, 24% removed — **J**, Forest Service.

(124) **A**, 1.25 — **B**, Scranton, Pa., Bellevue Mine — **C**, 2 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, no decay — **I**, 50% removed, not for decay — **J**, Forest Service.

(125) **A**, .55 — **B**, Scranton, Pa., Bellevue Mine — **C**, 3 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, no decay — **J**, Forest Service.

(126) **A**, 2.3 — **B**, Scranton, Pa., Dodge Coll. Drift — **C**, 1 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, decayed and attacked by insects — **J**, Forest Service.

(127) **A**, 4.3 — **B**, Scranton, Pa., Dodge Coll. Drift — **C**, 12 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, 67% sound, 25% slt. decayed — **I**, 8% broken or split — **J**, Forest Service.

(128) **A**, 2.6 — **B**, Scranton, Pa., Dodge Coll. Drift — **C**, 9 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, 78% sound, 11% slt. decayed — **J**, Forest Service.

(129) **B**, Scranton, Pa., Dodge Coll. Drift — **C**, 13 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, 77% sound, 23% split — **I**, no decay — **J**, Forest Service.

(130) **B**, Scranton, Pa., Dodge Coll. Drift — **C**, 3 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, 67% sound, 33% split — **I**, no decay — **J**, Forest Service.

(131) **A**, 3.0 — **B**, Scranton, Pa., Bellevue Mines — **C**, 3 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, no decay — **J**, Forest Service.

(132) **A**, 6.0 — **B**, Scranton, Pa., Bellevue Mines — **C**, 5 — **D**, 1908 — **E**, 4 — **F**, 1912 — **G**, no decay — **J**, Forest Service.

(133) **B**, Auchincloss Mine, Baltimore Vein Gangway — **D**, 1908 — **E**, 6 — **F**, 1914 — **G**, good — **J**, C. V. Ray, Ch. Eng., D.L. & W.R.R.

(133-A) **A**, 4-6 — **B**, Pottsville, Pa., P. & R. Mines — **C**, 138 — **D**, 1906 — **E**, 4 — **F**, 1910 — **G**, 75% sound, none decayed — **H**, crush, squeeze, etc. — **I**, treated green — 25% removed — **J**, Forest Service.

(133-B) **A**, 10-12 — **B**, Pottsville, Pa., P. & R. Mines — **C**, 135 — **D**, 1906 — **E**, 4 — **F**, 1910 — **G**, 72% sound, none decayed — **H**, crush, squeeze, etc. — **I**, treated seas., 28% removed — **J**, Forest Service.

(133-C) **A**, 4-6 — **B**, Pottsville, Pa., P. & R. Mines — **C**, 147 — **D**, 1907 — **E**, 3 — **F**, 1910 — **G**, 88% sound, 3% show decay — **H**, decay, crush, squeeze, etc. — **I**, treated green, 9% removed — **J**, Forest Service.

(133-D) **A**, 10-12 — **B**, Pottsville, Pa., P. & R. Mines — **C**, 36 — **D**, 1907 — **E**, 3 — **F**, 1910 — **G**, 80% sound, 9% show decay — **H**, decay, crush, squeeze, etc. — **I**, treated seas., 11% removed — **J**, Forest Service.

(133-E) **A**, 0.5 — **B**, Pottsville, Pa., P. & R. Mines — **C**, 48 — **D**, 1906 — **E**, 4 — **F**, 1910 — **G**, 60% sound, 15% show decay — **H**, decay, crush, squeeze, etc. — **I**, treated green, 25% removed — **J**, Forest Service

(133-F) **A**, 0.5 — **B**, Pottsville, Pa., P. & R. Mines — **C**, 129 — **D**, 1906 — **E**, 4 — **F**, 1910 — **G**, 80% sound, none decayed — **H**, crush, squeeze, etc. — **I**, treated seas., 20% removed — **J**, Forest Service.

(133-G) **A**, 0.3 — **B**, Pottsville, Pa., P. & R. Mines — **C**, 87 — **D**, 1906 — **E**, 4 — **F**, 1910 — **G**, 78% sound, none decayed — **H**, crush, squeeze, etc. — **I**, 22% removed — **J**, Forest Service.

Poles.

(134) B, Los Angeles, Cal. — C, 58 — D, 1907-8 — E, 5-6 — F, 1913 — G, 46.5% show decay — J, Forest Service.

(135) B, Los Angeles, Cal. — C, 11 — D, 1907-8 — E, 5-6 — F, 1913 — G, 81.8% show decay — J, Forest Service.

(136) Cal., Stokes Mt. line — C, 44 — D, 1908 — E, 5-6 — F, 1914 — G, 95.5% show decay — J, Forest Service.

(137) B, N. Y., Warren-Buffalo line — C, 66 — D, 1905 — E, 8 — F, 1913 — G, 39.0% show decay — J, Forest Service.

(138) B, Savannah, Ga. — C, 18 — D, 1905 — E, 8 — F, 1913 — G, 11% removed — H, decay — J, Forest Service.

(139) B, Savannah, Ga. — C, 24 — D, 1905 — E, 8 — F, 1913 — G, 29% removed — H, decay — J, Forest Service.

(140) B, Savannah, Ga. — C, 4 — D, 1905 — E, 8 — F, 1913 — G, 75% show decay — J, Forest Service.

(141) B, N.Y., Warren-Buffalo line — C, 8 — D, 1905 — E, 8 — F, 1913 — G, all show decay — J, Forest Service.

(142) B, Savannah, Ga. — C, 24 — D, 1905 — E, 8 — F, 1913 — G, 13% removed — H, decay — J, Forest Service.

(143) B, N. Y., Warren-Buffalo line — C, 63 — D, 1905 — E, 8 — F, 1913 — G, 94% show decay — J, Forest Service.

(144) B, Savannah, Ga. — C, 24 — D, 1905 — E, 8 — F, 1913 — G, 38% removed — H, decay — J, Forest Service.

(145) B, N. Y., Warren-Buffalo line — C, 28 — D, 1905 — E, 8 — F, 1913 — G, 4% show decay — J, Forest Service.

(146) N. Y., Warren-Buffalo line — C, 153 — D, 1905 — E, 8 — F, 1913 — G, 4% show decay — J, Forest Service.

(147) B, Cal., Stokes Mt. line — C, 236 — D, 1908 — E, 6 — F, 1914 — G, 5% show decay — J, Forest Service.

(148) B, Cal., near Los Angeles — C, 428 — D, 1907-8 — E, 5-6 — F, 1913 — G, 1.42% show decay — J, Forest Service.

(149) **B**, Cal., near Los Angeles — **C**, 28 — **D**, 1907-8 — **E**, 5-6 — **F**, 1913 — **G**, all sound — **J**, Forest Service.

(150) **B**, Savannah, Ga. — **C**, 6 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 17% removed — **H**, decay — **J**, Forest Service.

(151) **B**, Savannah, Ga. — **C**, 14 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 30% removed — **H**, decay — **J**, Forest Service.

(152) **B**, N. Y., Warren-Buffalo line — **C**, 83 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 37% show decay — **J**, Forest Service.

(153) **B**, Savannah, Ga. — **C**, 39 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 11% removed — **H**, decay — **J**, Forest Service.

(154) **B**, Savannah, Ga. — **C**, 34 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 19% removed — **H**, decay — **J**, Forest Service.

(155) **B**, Cal., Stokes Mt. line — **C**, 47 — **D**, 1908 — **E**, 6 — **F**, 1914 — **G**, 97.8% show decay — **J**, Forest Service.

(156) **B**, Los Angeles, Cal. — **C**, 65 — **D**, 1907-8 — **E**, 5-6 — **F**, 1913 — **G**, 44.6% show decay — **J**, Forest Service.

(157) **A**, 12 — **B**, Norfolk, Washington — **C**, 9876 — **D**, 1897 — **E**, 17 — **F**, 1914 — **G**, of 1600 inspected, 15.9% were replaced — **H**, see remarks — **I**, of the 15.9% replaced, 2.25% were due to decay — **J**, A. T. & T. letter of Feb. 10, 15, R. F. Hosford.

(157-A) **A**, 12 — **B**, Montgomery-New Orleans — **C**, 7644 — **D**, 1899 — **E**, 15 — **F**, 1914 — **G**, total removals less than 5% — **H**, large portion due to losses in storms — **J**, A.T. & T., letter of Feb. 10, 15, R. F. Hosford.

(158) **B**, Stokes Mt. line — **C**, 13 — **D**, 1908 — **E**, 6 — **F**, 1914 — **G**, 30.7% show decay — **J**, A.T. & T., R. F. Hosford, letter Feb. 10, 15.

(159) **B**, near Los Angeles — **C**, 3 — **D**, 1907-8 — **E**, 5-6 — **F**, 1913 — **G**, 66.6% show decay — **J**, A.T. & T., R. F. Hosford, letter Feb. 10, 15.

(160) **B**, Savannah, Ga. — **C**, 24 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 25% removed — **H**, decay — **J**, Forest Service.

(161) **B**, Savannah, Ga. — **C**, 24 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 48% removed — **H**, decay — **J**, Forest Service.

(162) **B**, N. Y., Warren-Buffalo line — **C**, 67 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 51% show decay — **J**, Forest Service.

(163) **B**, Savannah, Ga. — **C**, 18 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 12% removed — **H**, decay — **J**, Forest Service.

(164) **B**, Savannah, Ga. — **C**, 21 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 42% removed — **H**, decay — **J**, Forest Service.

(165) **B**, Savannah, Ga. — **C**, 3 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 33% removed — **H**, decay — **J**, Forest Service.

(166) **B**, Savannah, Ga. — **C**, 3 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, all show decay — **J**, Forest Service.

(167) **B**, N. Y., Warren-Buffalo line — **C**, 10 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 80% show decay — **J**, Forest Service.

(168) **B**, Savannah, Ga. — **C**, 24 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 35% removed — **H**, decay — **J**, Forest Service.

(169) **B**, Savannah, Ga. — **C**, 24 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 13% removed — **H**, decay — **J**, Forest Service.

(170) **B**, N. Y., Warren-Buffalo line — **C**, 48 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, all show decay — **J**, Forest Service.

(171) **B**, Savannah, Ga. — **C**, 8 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 12% removed — **H**, decay — **J**, Forest Service.

(172) **B**, Savannah, Ga. — **C**, 21 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 34% removed — **H**, decay — **J**, Forest Service.

(173) **B**, N. Y., Warren-Buffalo line — **C**, 50 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, 58% show decay — **J**, Forest Service.

(174) **B**, N. Y., Warren-Buffalo line — **C**, 22 — **D**, 1905 — **E**, 8 — **F**, 1913 — **G**, all show decay — **J**, Forest Service.

(175) **B**, Cal., near Los Angeles — **C**, 163 — **D**, 1907-8 — **E**, 5-6 — **F**, 1913 — **G**, 58.9% show decay — **J**, Forest Service.

(176) **B**, Cal., Stokes Mt. line — **C**, 42 — **D**, 1908 — **E**, 6 — **F**, 1914 — **G**, 50% show decay — **J**, Forest Service.

(177) **B**, Cal., near Los Angeles — **C**, 51 — **G**, 17.6% show decay — **J**, Forest Service.

Posts.

(178) **B**, Zumbra Heights, Minn. — **C**, 481 — **D**, 1908 — **E**, 4.5 — **F**, 1913 — **G**, 0.6% removed — **H**, decay — **I**, entire post treated — **J**, Forest Service.

(179) **B**, Calhoun, La. — **C**, 97 — **D**, 1908-10 — **E**, 4.5 — **F**, 1912 — **G**, 14.4% decayed and missing — **J**, Forest Service.

(180) **B**, Ames, Iowa — **C**, 288 — **D**, 1909 — **E**, 4 — **F**, 1913 — **G**, good — **J**, Forest Service.

(181) **B**, Calhoun, La. — **C**, 48 — **D**, 1908-10 — **E**, 4.5 — **F**, 1912 — **G**, good — **J**, Forest Service.

(182) **B**, Calhoun, La. — **C**, 93 — **D**, 1908-10 — **E**, 4.5 — **F**, 1912 — **G**, 1.1% removed — **H**, decay — **J**, Forest Service.

(183) **B**, Calhoun, La. — **C**, 50 — **D**, 1908-10 — **E**, 4.5 — **F**, 1912 — **G**, 10.0% decayed and missing — **J**, Forest Service.

(184) **B**, Zumbra Heights, Minn. — **C**, 254 — **D**, 1908 — **E**, 4.5 — **F**, 1913 — **G**, 0.4% decayed — **I**, entire post treated — **J**, Forest Service.

(185) **B**, Zumbra Heights, Minn. — **C**, 59 — **D**, 1908 — **E**, 4.5 — **F**, 1913 — **G**, good — **I**, butts only treated — **J**, Forest Service.

(186) **B**, Clemson College, S. C. — **C**, 254 — **D**, 1909 — **E**, 3 — **F**, 1912 — **G**, good — **J**, Forest Service.

(187) **B**, Auburn, Ala. — **C**, 54 — **D**, see remarks — **E**, 5.5 — **F**, 1913 — **G**, 76% good — **H**, decay — **I**, piled 2 years before setting, in ground 3.5 years — **J**, Forest Service.

(188) **B**, Calhoun, La. — **C**, 312 — **D**, 1908-10 — **E**, 4.5 — **F**, 1912 — **G**, 4.5% decayed and missing — **H**, decay — **I**, piled 2.5 years before setting, in ground 2.5-4.5 years — **J**, Forest Service.

(189) **B**, Clemson College, S. C. — **C**, 163 — **D**, 1909 — **E**, 3 — **F**, 1912 — **G**, 0.6% removed — **H**, decay — **J**, Forest Service.

(190) **B**, Clemson College, S. C. — **C**, 254 — **D**, 1909 — **E**, 3 — **F**, 1912 — **G**, 0.4% removed — **H**, decay — **J**, Forest Service.

(191) **B**, Ames, Iowa — **C**, 49 — **D**, 1909 — **E**, 4 — **F**, 1913 — **G**, good — **H**, decay — **J**, Forest Service.

(192) **B**, Ames, Iowa — **C**, 45 — **D**, 1909 — **E**, 4 — **F**, 1913 — **G**, good — **H**, decay — **J**, Forest Service.

Paving Blocks.

(193) **B**, Seattle, Wash. — **C**, 1144 square yards — **D**, 1897 — **F**, 1914 — **G**, replaced — **H**, not stated — **J**, Seattle, Wash., City Eng.

(194) **B**, Seattle, Wash. — **C**, 1227 square yards — **D**, 1902 — **E**, 12 — **F**, 1914 — **G**, part replaced — **H**, decay — **J**, Seattle, Wash., City Eng.

(195) **A**, 20.0 — **B**, Birmingham, Ala. — **C**, 2700 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, fairly good — **J**, Birmingham, Ala., City Eng.

(196) **A**, 20.0 — **B**, Mobile, Ala. — **C**, 13,023 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, good — **J**, Mobile, Ala., City Eng.

(197) **A**, 20.0 — **B**, Mobile, Ala. — **C**, 11,158 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, fair — **I**, 3 inch blocks, sand filler — **J**, Mobile, Ala., City Eng.

(198) **A**, 16.0 — **B**, Kansas City, Mo. — **C**, 6860 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, fair — **J**, Kansas City, Mo., Asst. City Eng.

(199) **A**, 10-12 — **B**, Indianapolis, Ind. — **C**, 60,439 square yards — **D**, 1898 — **E**, 16 — **F**, 1914 — **G**, bad — **J**, Indianapolis, Ind., City Eng.

(200) **A**, 16 — **B**, Minneapolis, Minn. — **C**, 44,838 square yards — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, fair — **J**, Minneapolis, Minn., City Eng.

(201) **A**, 16 — **B**, Minneapolis, Minn. — **C**, 74,358 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, fair — **J**, Minneapolis, Minn., City Eng.

(202) **A**, 16-22 — **B**, Minneapolis, Minn. — **C**, 1369 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, see remarks — **H**, Douglas fir blocks removed because of inferior material used — **I**, Douglas fir removed in 1911 replaced with new blocks. Longleaf pine and white birch sections in excellent condition, others fair — **J**, Minneapolis, Minn., City Eng.

(203) **A**, 20 — **B**, Sioux City, Iowa — **C**, 16,120 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, see remarks — **I**, trouble from buckling, otherwise good — **J**, Sioux City, Iowa, City Eng.

(204) **A**, 16 — **B**, La Crosse, Wis. — **C**, 1925 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, fair — **J**, La Crosse, Wis., City Eng.

(205) **B**, Duluth, Minn. — **C**, 15,940 square yards — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, good — **I**, treated in accordance with Spec. of A.S. P.S. — **J**, Duluth, Minn., City Eng.

(206) **A**, 20 — **B**, Birmingham, Ala. — **C**, 6138.7 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, poor — **J**, Birmingham, Ala., City Eng.

(207) **A**, 22 — **B**, Springfield, Mass. — **C**, 5652 square yards — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, fair — **I**, rough in spots — **J**, Springfield, Mass., City Eng.

(208) **A**, 22 — **B**, Springfield, Mass. — **C**, 11,981 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, fair — **I**, rough in spots — **J**, Springfield, Mass., City Eng.

(209) **A**, 20 — **B**, Springfield, Mass. — **C**, 6460 square yards — **D**, 1905 — **E**, 8 — **F**, 1914 — **G**, fair — **I**, rough in spots — **J**, Springfield, Mass., City Eng.

(210) **B**, Steelton, Pa. — **C**, 32,831 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, good — **J**, Steelton, Pa., City Eng.

(211) **A**, 12 — **B**, Minneapolis, Minn. — **C**, 13,616 square yards — **D**, 1902 — **E**, 12 — **F**, 1914 — **G**, excellent — **J**, Minneapolis, Minn., City Eng.

(212) **A**, 12 — **B**, Minneapolis, Minn. — **C**, 300 square yards — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, excellent — **J**, Minneapolis, Minn., City Eng.

(213) **A**, 16 — **B**, Toledo, Ohio — **C**, 6601 square yards — **D**, 1902 — **E**, 12 — **F**, 1914 — **G**, good — **J**, Toledo, Ohio, City Eng.

(214) **A**, 12 — **B**, Minneapolis, Minn. — **C**, 30,525 square yards — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, fair — **J**, Minneapolis, Minn., City Eng.

(215) **A**, 12 — **B**, Minneapolis, Minn. — **C**, 59,186 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, fair — **J**, Minneapolis, Minn., City Eng.

(216) **B**, Indianapolis, Ind. — **C**, 7769 square yards — **D**, 1902 — **E**, 12 — **F**, 1914 — **G**, fair — **J**, Indianapolis, Ind., City Eng.

(217) **B**, Chicago, Ill. — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, fair — **J**, Chicago, Ill., City Eng.

(218) **B**, Chicago, Ill. — **D**, 1899 — **E**, 7 — **F**, 1906 — **G**, fair — **I**, Rush Street Bridge pavement removed but not on account of decay — **J**, Chicago, Ill., City Eng.

(219) **B**, Chicago, Ill. — **C**, 3119 square yards — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, fair — **J**, Chicago, Ill., City Eng.

(220) **B**, Chicago, Ill. — **C**, 13,520 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, fair — **J**, Chicago, Ill., City Eng.

(221) **B**, Indianapolis, Ind. — **C**, 7863 square yards — **D**, 1899 — **E**, 15 — **F**, 1914 — **G**, bad — **J**, Indianapolis, Ind., City Eng.

(221) **B**, Indianapolis, Ind. — **C**, 19,916 square yards — **D**, 1900 — **E**, 14 — **F**, 1914 — **G**, bad — **J**, Indianapolis, Ind., City Eng.

(221) **B**, Indianapolis, Ind. — **C**, 35,774 square yards — **D**, 1901 — **E**, 13 — **F**, 1914 — **G**, fair — **J**, Indianapolis, Ind., City Eng.

(221) **B**, Indianapolis, Ind. — **C**, 8678 square yards — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, fair — **J**, Indianapolis, Ind., City Eng.

(221) **B**, Indianapolis, Ind. — **C**, 24,612 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, fair — **J**, Indianapolis, Ind., City Eng.

(221) **B**, Indianapolis, Ind. — **C**, 13,112 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, good — **J**, Indianapolis, Ind., City Eng.

(222) **A**, 20 — **B**, Boston, Mass. — **C**, 1295 square yards — **D**, 1900 — **E**, 14 — **F**, 1914 — **G**, excellent — **J**, Boston, Mass., Chief Eng.

(222) **A**, 20 — **B**, Boston, Mass. — **C**, 11,684 square yards — **D**, 1901 — **E**, 13 — **F**, 1914 — **G**, very good — **J**, Boston, Mass., Chief Eng.

(222) **A**, 20 — **B**, Boston, Mass. — **C**, 2859 square yards — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, very good — **J**, Boston, Mass., Chief Eng.

(223) **A**, 12 — **B**, St. Louis, Mo. — **C**, 49,614 square yards — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, good — **I**, some decay in center of blocks — **J**, St. Louis, Mo., Chief Eng.

(224) **B**, New York City, Boro of Brooklyn — **C**, 1152 square yards — **D**, 1902 — **E**, 12 — **F**, 1914 — **G**, very good — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Brooklyn — **C**, 13,555 square yards — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, very good — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Brooklyn — **C**, 20,016 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, very good — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Brooklyn — **C**, 4200 square yards — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, good — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Queens — **C**, 1230 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, repaved — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Queens — **C**, 60,626 square yards — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, good — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Bronx — **C**, 8101.6 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, fair to poor — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Bronx — **C**, 321 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, good — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Manhattan — **C**, 20,902 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, repaved — **I**, not due to decay — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Manhattan — **C**, 49,251 square yards — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, good — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Manhattan — **C**, 92,808 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, good — **J**, New York City, Sec. to Consult. Eng.

(224) **B**, New York City, Boro of Richmond — **C**, 3471 square yards — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, good — **J**, New York City, Sec. to Consult. Eng.

(225) **B**, Jamestown, N. Y. — **C**, 1500 square yards — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, good — **I**, some trouble on account of slipperiness and expansion probably creo-resinate — **J**, Jamestown, N. Y., City Eng.

(226) **B**, Geneva, N. Y. — **C**, 1606 square yards — **D**, 1900 — **E**, 14 — **F**, 1914 — **G**, very good — **I**, trouble from expansion — **J**, Geneva, N. Y., City Eng.

(227) **B**, Fargo, N. D. — **C**, 2900 square yards — **D**, 1901 — **E**, 13 — **F**, 1914 — **G**, good — **I**, bridge flooring — **J**, Fargo, N. D., City Eng.

(228) **B**, Harrisburg, Pa. — **C**, 3006 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, good — **J**, Harrisburg, Pa., City Eng.

(229) **B**, Lexington, Ky. — **C**, 8000 square yards — **D**, 1904 — **E**, 10 — **F**, 1914 — **G**, excellent — **J**, Lexington, Ky., City Eng.

(230) **B**, Seattle, Wash. — **C**, 10 square yards — **D**, 1903 — **E**, 11 — **F**, 1914 — **G**, part replaced — **J**, Seattle, Wash., City Eng.

(231) **B**, Boston, Mass. — **C**, 10,704 square yards — **D**, 1906 — **E**, 8 — **F**, 1914 — **G**, good — **J**, Boston, Mass., City Eng.

(232) **B**, Houghton, Mich. — **C**, 13,000 square yards — **D**, 1905 — **E**, 9 — **F**, 1914 — **G**, good — **J**, Houghton, Mich., City Eng.

Miscellaneous.

(233) **A**, 15 — **B**, New York City, 5th Ave. — **D**, 1887 — **E**, 21 — **F**, 1908 — **G**, sound — **I**, removed for placing of tunnel — **J**, Wyckoff Pipe & Creo. Co.

(234) **B**, New Orleans — **D**, 1889 — **E**, 21 — **F**, 1910 — **G**, sound — **I**, flooring in contact with ground — **J**, Mr. Young, letter from L. B. Shipley, Barrett Mfg. Co.

DISCUSSION

Mr. F. J. Angier* considers the chemical treatment of timber a very important factor in the conservation of our natural resources. The wood-preserving industry is growing and the railroads are gradually building plants to preserve their cross ties and other timber which is subject to decay. Mr. Angier.

He states that the treatment of ties by railroads is generally considered for two reasons: First, the serious problem and steady increase in tie expense; and second, the desire to co-operate in conserving the timber supply of the country. The amount spent on the Baltimore & Ohio Railroad for ties in 1913 was over $2,200,000. This does not include ties for construction work or cost of labor putting in track. It is safe to say that every other railroad in the United States is spending a proportionate amount for this one item of cross ties. With the exception of fuel, the expenditure for cross ties represents the largest single item of expense for material on a railroad.

He feels that timber preservation is really in its infancy, and doubtless will grow rapidly in succeeding years. The managers of railroads want to know what life can be obtained from treated timber. This information Messrs. Weiss and Teesdale have presented in a concise form and are to be congratulated in presenting such a valuable paper on the life of treated timber. It shows an immense amount of work in the collection of so much data from so many different sources, and it certainly will have a very wholesome effect on the timber preservation industry.

* Superintendent Timber Preservation, The Baltimore & Ohio System, Baltimore, Md.

Paper No. 100

CLAY PRODUCTS AS AN ENGINEERING MATERIAL.*

By

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GENERAL CONSIDERATIONS.

(a) Factors Governing the Properties of Clay Products.

In the discussion of the strength of burned clay, two points of view suggest themselves. One involves the intrinsic strength of the material for different degrees of hardness, i. e., with the progress of vitrification; the other, the strength of clay when fabricated into various shapes. The first case concerns itself with the fundamental properties of the material; the latter, with the structural aspect.

The treatment of the first case makes necessary, as much as possible, the elimination of all extraneous factors, such as unsymmetrical shape and molding structure. In the second case, the element of the shape of the product is introduced, as well as the molding structure, which is unavoidable and which may prove an important factor.

From the structural standpoint clay products cannot be studied as a material, but must be taken up according to the kind of product, whether solid or perforated bricks, building tile, architectural terra cotta, sewer pipe, fire-brick, conduits, etc.

The physical properties of clay products depend primarily upon the kind of clay from which they are manufactured. Since clays differ widely in chemical composition, in plasticity, fineness of grain, shrinkage, burning behavior, refractoriness, color, etc., according to their mineral structure and geological origin,

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the products made from them are likewise subject to wide variations as to their properties and appearance.

(b) Classification of Clays.

From the industrial standpoint, clays may be roughly divided into six classes:

1. Primary kaolins, usually white burning, very refractory, and of low plasticity and strength.

2. Plastic kaolins, not as white burning as primary clays. May be very plastic.

3. Clays of the fire-clay type, ranging from materials of high refractoriness to those inferior in this respect. Certain highly refractory clays, called flint clays, are almost lacking in plasticity. Very plastic clays, called ball clays.

4. Shales. Fine-grained, hard clays, showing well-defined cleavage and often slate-like in appearance. They differ widely as to composition. Many ferruginous shales are sources of best paving brick. Shales, as a rule, are uniform over comparatively large areas, and hence have great industrial importance.

5. Alluvial clays. Of recent geological origin and found principally in old and new river valleys. Vary widely in composition and physical behavior, from clays of excellent plasticity to very sandy materials. As a rule, unsuitable for paving brick. Usually burn to a red color.

6. Glacial clays. Heterogeneous mixture of clay with various rock debris. Frequently calcareous, causing clay to burn to a cream or buff color. As a class, difficult to work and hence suitable only for crude products.

7. Loess clays. Wind-blown deposits, usually silicious and hence of low plasticity. Used only for common bricks.

From the structural standpoint, the No. 2 fire clays, shales, and alluvial clays are of greatest importance.

(c) Classification of Clay Products.

For the purpose of the present paper eight groups of clay products may be considered, as follows:

1. Building brick.
2. Paving brick.
3. Hollow building tile and fire-proofing.
4. Architectural terra cotta.
5. Sewer pipe and drain tile.

6. Refractories.
7. Floor, wall and roofing tile, glazed brick.
8. Sanitary ware.

According to the particular use of each class of product, certain requirements must be emphasized, which suppose the predominance of definite physical qualities and the minor consideration of others. These various physical properties might be summarized as follows:

1. Compressive strength.
2. Transverse strength.
3. Modulus of elasticity.
4. Resistance to shear.
5. Hardness and toughness.
6. Porosity and resistance to weathering and corrosion.
7. Resistance to high temperatures and sudden temperature changes.
8. Thermal conductivity, specific heat and thermal expansion.
9. Sanitary aspect.

The subject can be treated most expeditiously by taking up the various clay products in the order named above.

BUILDING BRICK.

(a) Relation Between Intrinsic and Structural Strength.

The principal requirements with reference to building brick are compressive strength and resistance to transverse rupture, and porosity, together with durability, i. e., resistance to weathering influences. It must be realized, however, that from the very nature of the material the stresses to which it is subjected in structures are but a small fraction of the intrinsic strength. This is due to the fact that the mortar is an important factor. Thus, no matter how hard and strong the bricks may be, if, for instance, lime mortar is used, the compression strength of the structure will not exceed a certain limit. The stronger the mortar, the more will the strength of the walls or piers approach that of the individual brick. This has been shown very conclusively by the pier tests made at the Watertown Arsenal and the Bureau of Standards.

(b) Intrinsic Compressive Strength of Clays.

The intrinsic strength of clays has been recently determined at the Bureau of Standards for 3.5-inch clay cubes, made from typical clay deposits and burned to several temperatures. These results are shown graphically in the diagram of Fig. 1, from which it will be seen that with some clays the rate of increase in strength, from the point of maximum porosity, is quite marked at the beginning and then remains practically

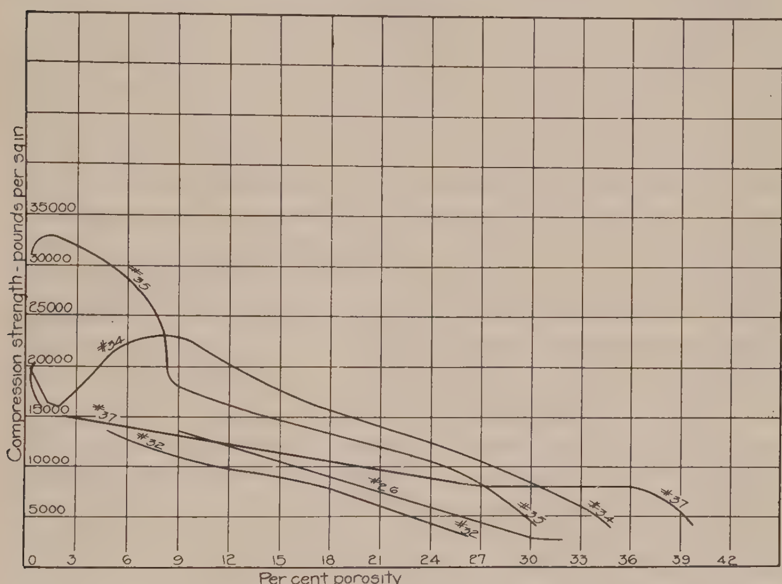

Fig. 1. Variation of Compressive Strength with Porosity. Bureau of Standards Tests.

stationary for a considerable interval. Others show a steady increase. The state of vitrification is characterized, as a rule, by a decided rate of gain in strength. This is due principally to the gain in the inherent, mechanical strength of the solid vitrified clay and only secondarily to the decrease in pore space. Overburning, with its accompanying vesicular structure, is at once indicated by a drop in strength. The porosity, *per se*, affords a good index of the quality of the bricks made from a given clay, but cannot be used as a general criterion for all

kinds of clays. This is shown by the following results, which give the porosity of six clays at the point at which all showed a compressive strength of 5000 pounds per square inch.

Sample No.	32	26	33
Kind of clay	Fire clay	Ferruginous surface clay	No. 2 fire clay
Per cent porosity	23	25	26.5

Sample No.	35	34	37
Kind of clay	Shale	Shale	Calcareous surface clay
Per cent porosity	29.5	33.5	40

The porosity, in these examples, represents the percentage of the actual pore space in terms of the volume of the specimen. Usually, the porosity is estimated by means of the absorption test, which is made by immersing half-bricks in water for twenty-four hours and boiling them for four hours. The difference between the weights of the dry and the saturated specimens (weighed cold), computed in per cent of the dry weight, gives the absorption. Approximately, per cent porosity corresponds to twice the absorption values.

The compressive strength reached necessarily varies with the kind of clay. Shales, as a rule, result in the greatest strengths; calcareous glacial clays (not vitrified), in the lowest. Hard-fired, red-burning alluvial clays are about equivalent in this respect to the corresponding grade of building brick made from the semi-refractory fire clays.

The results of the compressive strength-porosity relations are compiled in Table I.

(c) Compressive Strength of Bricks.

With reference to the compressive strength of commercial bricks, it must be realized that the strength is not uniform in all directions. According to the method of manufacture, it will be greater along one plane than along any others, according to the direction of the flow of the material in the plastic state. In addition, the question of such structure as auger lamination enters as an important factor. It is a well-known fact that structure is produced, in shaping the brick, by means

Table I.

No. of clay	Firing temperature in cones	Per cent porosity	Average compression strength, lbs. per sq. in.	Average per cent deviation from mean crushing strength
26	08	30.94	2,281	8.10
	06	31.86	2,944	9.54
	04	27.37	4,085	14.48
	1	20.56	7,467	7.91
	3	21.47	8,101	16.57
	5	9.13	13,368	3.25
32	08	26.23	3,014	10.02
	06	24.11	4,217	15.64
	02	16.00	8,485	6.13
	3	16.08	9,205	11.49
	5	10.29	10,339	8.83
	7	4.79	13,424	18.10
33	08	27.23	4,139	11.61
	06	25.95	4,883	8.72
	04	22.28	9,335	5.32
	02	21.74	9,318	8.45
	1	20.47	10,555	14.25
	5	14.47	9,751	22.37
	7	12.12	15,353	12.69
34	08	34.74	3,949	10.69
	06	33.34	5,795	12.74
	04	21.95	13,538	8.55
	02	16.74	16,368	8.38
	1	15.22	17,340	11.33
	3	6.45	22,583	9.50
	5	1.41	15,491	27.45
	7	0.30	19,362	9.19
35	08	30.31	4,111	14.10
	06	26.42	8,967	14.50
	04	17.69	13,989	15.00
	02	8.53	18,511	7.50
	1	7.94	24,399	14.36
	3	0.66	32,341	19.84
	5	0.16	30,342	17.08
37	06	40.51	4,136	7.79
	02	38.14	6,668	6.77
	1	35.26	7,887	8.41
	3	27.22	7,887	16.89
	5	0.42	16,895	15.56
	7	0.26	20,283	8.04

of the auger machine, both by the action of the auger and by the differential flow of the clay through the die. The clay is delivered from the auger wings in helical strands, and in this condition is forced through the die. The strands of the clay may be imperfectly welded, thus giving a structure similar to the twisted strands of a rope. On the other hand, the friction of the clay against the iron surface of the die causes the clay to flow at a greater speed at the center of the stream, resulting in a structure resembling overlapping cones. This is called differential lamination.

Dry-pressed bricks, when hard burned, result in the greatest compression values, owing to their density. In well-burned building brick, lamination structure, unless very excessive, has no special practical importance. It does, however, contribute towards the variable results obtained in testing. In this respect, the shape of the test specimen is of primary importance. Theoretical considerations show that the cube, or still better, the prism of square cross section, the height of which is one and one-half times the breadth, gives best results, which is shown by the symmetry of the fracture planes. If a brick is tested flatwise, in which position the ratio $b_1 \div h = 2$, the resulting compressive strength will be 22 per cent greater than that of a cube made from the same material, and 33 per cent greater than that for the specimen in which $b_1 \div h = 0.66$. The practice of testing whole bricks flatwise is especially to be condemned, since the amount of side restraint in this case is very great and is certain to result in a fictitious value.

(d) Practice of Testing Bricks.

The common American practice is to test half-brick, bedded in plaster to secure parallel surfaces. It would be far more desirable to test two half-bricks, one cemented upon the top of the other, as is done in German practice; or, still better, to cement together three halves, so as to form a column of greater height. In this manner the side restraint of the machine heads would affect the top and bottom pieces, but would not affect the middle piece. The objection to this practice is, of course, the more laborious preparation of the specimens. A compromise might be suggested, viz., the testing of half-bricks on edge. While not as desirable as the method suggested above, this

manner of testing is greatly to be preferred to placing the bricks flatwise. It gives more consistent, sharper results, and is carried out even more easily.

The relation between these two kinds of tests was examined by determining the crushing strength of bricks* collected from about 15 brick plants within a radius of 150 miles from Washington, D. C. In this work the bricks were cut in two, one half having been tested flatwise, and the other on edge. The bricks tested were of the harder grades and nearly all of them had been made by the stiff-clay process. Both shale and clay bricks were represented, and 178 bricks were thus tested. The average compression strength of the bricks tested flatwise was 6226 pounds per square inch and of the bricks tested on edge 5399 pounds. The ratio was hence $6226 \div 5399 = 1.153$.

(e) Brick Specifications.

Any one familiar with the various grades of bricks used throughout the country must realize that, owing to the widely differing kinds of raw material, considerable variations prevail in regard to the physical strength of the products. The alluvial clay deposits of the Hudson river produce a different grade of brick from the calcareous glacial clays of the Chicago district, or the shales of Pennsylvania, Ohio, Indiana, West Virginia, and other States, or the surface and loess clays of the more western States. A discussion of this subject is necessary, therefore, in order that it may be possible to adopt specifications insuring to the user products of reasonable strength. It would not be fair to make no differentiation between a well-made and burned product and an inferior one, as is now the case in many places where the average contractor looks more to cheapness than quality. By recognizing quality, and thus offering the possibility of a better price for better products, an incentive is furnished the manufacturer, which is lacking at the present time. One phase of the subject to be mentioned is its legal side. Definite requirements for bricks and similar products are necessary, in order that legal stipulations concerning quality may possess a well-defined basis.

These facts have been realized by the Committee on Building Brick of the American Society for Testing Materials, and

* Trans. Am. Ceram. Soc., 12, 564.

in their report of 1913* four classes of bricks are recognized, with the following limits for crushing strength and absorption:

Class A. Vitrified brick. Average compressive strength not less than 5000 pounds per square inch; average absorption not more than 5 per cent.

Class B. Hard-burned brick. Average compressive strength 3500-5000 pounds per square inch; average absorption not more than 12 per cent.

Class C. Common brick. Average compressive strength 2000-3500 pounds per square inch; average absorption not more than 18 per cent.

Class D. Common brick, second, not to be used in exposed work. Average compressive strength not less than 1500 pounds per square inch. No limits are set as regards absorption.

(f) Tests of Bricks from Several Districts.

In cooperation with the above Committee, twelve testing laboratories undertook to test bricks collected from ten States. It was found in this work that the absorption values could not be used as a means of differentiating the different classes of brick; hence, these were discarded for this purpose. The general averages obtained for the compressive strength were as follows: Class A, 8493; Class B, 4171; Class C, 2791; Class D, 1762.

From extensive tests carried on at the Department of Experimental Engineering of the University of Minnesota, on bricks made in that State, the average values for the different classes were found to be: Class A, 5663; Class B, 4762; Class C, 2584; Class D, 1739; Class E (strengths below class D), 907.

The results obtained by Prof. Ira A. Woolson, in connection with bricks from the Hudson River district used so largely in New York City, indicate a mean compressive strength of 3943 pounds per square inch.

Tests made by the Bureau of Standards on bricks obtained from 20 brick yards in the Chicago district showed an average crushing strength of 2156 pounds per square inch and an absorption of 23.79 per cent.

A series of bricks burned to different temperatures, tested by the same Bureau, collected from a typical plant in the Pittsburgh district, showed a very consistent relation between

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* Am. Soc. Testing Materials, 13, p. 286.

porosity and crushing strength which could be expressed by the empirical equation, $y = 6400 - 38.7.5\,x - 16.25\,x^2$, where $y =$ crushing strength in pounds per square inch, and $x =$ per cent absorption.

(g) Transverse Strength.

It is common practice in the testing of bricks to make both the transverse and crushing-strength tests. Undoubtedly, both of them are of value, but in some respects the transverse test is superior, inasmuch as bricks in actual use are subjected to transverse stresses in a very large measure. This test brings out the effect of inferior structure very sharply. It would seem that the transverse test would make possible a sharper differentiation of the various classes of bricks. There is, however, no direct connection between crushing strength and the modulus of rupture. Bricks with a high compressive strength may show a low modulus of rupture, and vice versa, as has been shown to be the case in the work of Prof. Ira H. Woolson in connection with building brick from New Jersey.

The general average values obtained in the transverse test by the laboratories cooperating with the Committee on Brick Tests of the American Society for Testing Materials were found to be as follows: Class A, modulus of rupture, 1482; Class B, 668; Class C, 621; Class D, 469. The results obtained in the test series of the Department of Experimental Engineering, University of Minnesota differentiated the different grades of bricks in the following manner: Class A, 1253; Class B, 926; Class C, 576; Class D, 403. A comparison between the modulus of rupture and the compressive strength is shown in the diagram of Fig. 2, presenting results obtained by Prof. Woolson on bricks from the New York district. It will be noted that the rate of increase in crushing strength is very much more rapid than that for the modulus of rupture and that, hence, no well defined relation exists between the two factors.

(h) Modulus of Elasticity.

Owing to the characteristic rigid structure of brick and stone materials, the elastic limit is not always observable on single specimens, though it may be observed on built-up structures. The Pittsburgh laboratory of the Bureau of Standards undertook the study of the intrinsic modulus of elasticity of a

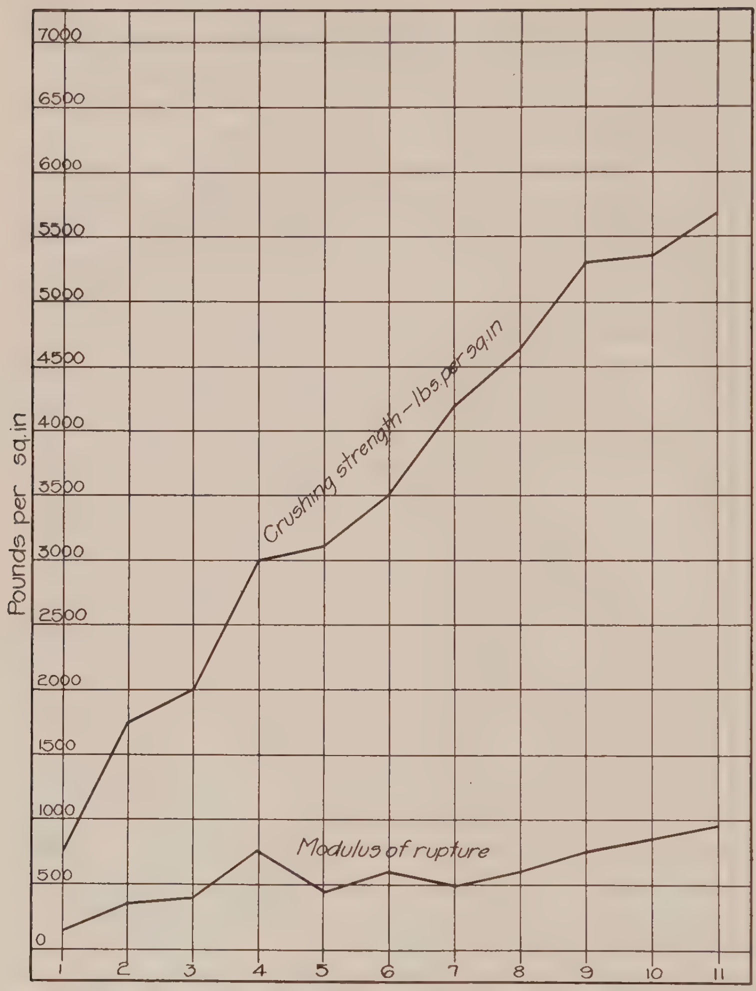

Fig. 2. Curves of Tests for Compressive Strength. Modulus of Rupture Relation.

number of typical clays. The size of the specimens was 3½ in. x 3½ in. x 12 in., the ends of which were ground parallel. In the majority of the stress-strain curves the inflexion point of the elastic limit failed to appear, due in part to the structure planes caused by the auger machine and in part to flexure of the column. The deformation, as expected, was found to be a

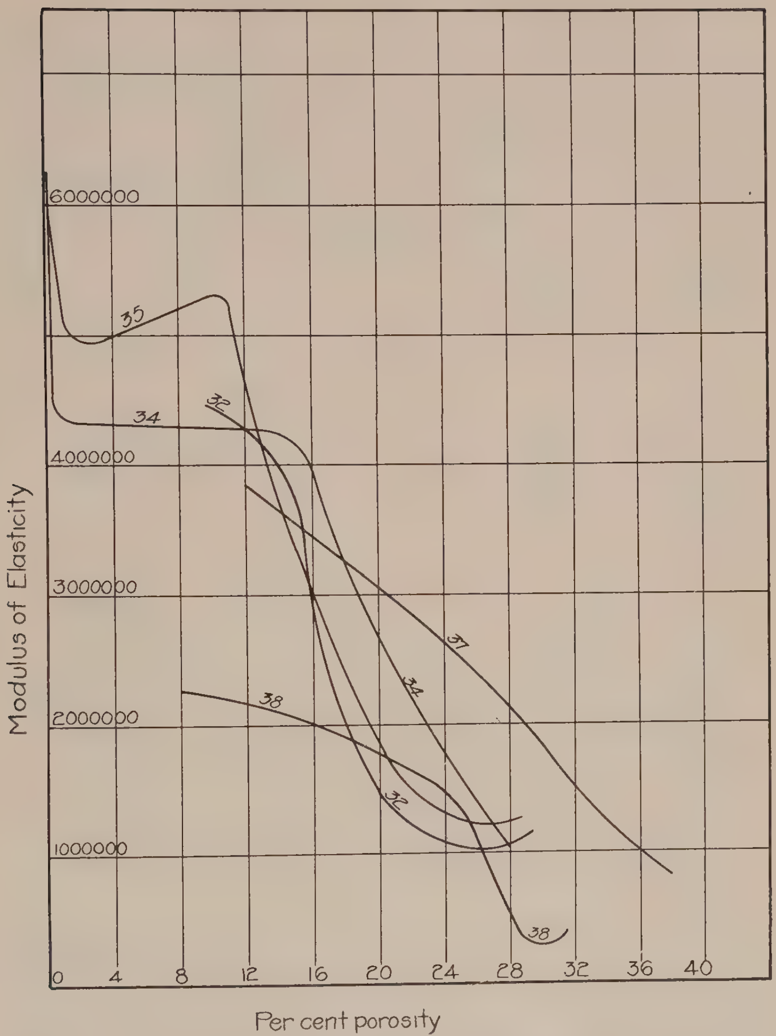

Fig. 3. Variation of Modulus of Elasticity with Porosity. Bureau of Standards Tests.

function of the porosity, as is shown in Fig. 3, in which the modulus of elasticity is plotted against the porosity. In the case of the shales it seems to be characteristic that the modulus increases gradually from the point of maximum porosity, re-

mains practically constant during the later interval of vitrification and rises rapidly as complete vitrification is approached. The No. 2 fire clay appears to show quite a regular increase in rigidity. The same relation in the case of the calcareous surface clay is almost linear and indicates that, when well prepared, this type of material possesses excellent structural properties. The other surface clay evidently possesses far lower rigidity. The highest moduli are shown by the shales, the lowest by one of the surface clays. These results are compiled in Table II.

Table II.

No. of Sample	Kind of Clay	Burning Temperature, in cones	Modulus of Elasticity
32	No. 2 fire clay	06	1,180,000
32	" " "	5	4,330,000
33	" " "	08	1,170,000
33	" " "	3	3,050,000
35	Shale	06	1,480,000
35	"	5	6,400,000
34	"	06	1,350,000
34	"	3	5,940,000
40	"	06	1,930,000
40	"	1	3,560,000
37	Surface clay (calcareous)	08	850,000
37	" " "	3	3,830,000
38	Surface clay (red burning)	06	376,000
38	" " " "	5	2,240,000

The marked differences in the value of the modulus of elasticity in favor of paving-brick shales calls attention to the several proposals found in the literature concerning the application of a function of this factor as regards the toughness of the materials. According to Johnson* "the area of the stress diagram up to the elastic limit, divided by the volume of the specimen under test is a measure of the ability of the material per unit volume, to absorb and give out energy or to resist repeated shocks without injury; and the total area of the stress

* J. B. Johnson, The Materials of Construction, p. 81.

diagram is to measure the ability to resist a single blow without rupture."

(i) Shearing Strength.

Results on shearing strengths of bricks are comparatively scarce. Some of the results obtained by J. E. Howard are compiled in the following table.†

Table III.

Kind of Brick	% Absorption	Shearing Strength, lbs. per sq. in.	Crushing Strength, lbs. per sq. in.
Dark red	....	1,011	10,643
Light buff	19.00	642	8,144
Red	14.80	784	5,589
Dark red	....	714	6,814
Buff	....	1,767	13,292
Buff	11.6	1,097	15,374
Light chocolate	14.5	988	13,059
Gray	15.2	536	3,070

(j) Hardness and Toughness.

For building brick, the function of hardness is identical with good crushing strength and is usually determined, in a practical way, by the ring of the bricks when knocked together, or by the cutting effect of a hardened-steel tool. The determination of the hardness by exact means is scarcely ever necessary, nor is toughness a quality specially looked for in building brick. The standardized sand-blast test* affords a satisfactory measure of hardness where it seems desirable to emphasize this quality.

(k) Brick Piers.

Howard's Work. The subject of brick piers has been given considerable attention in this country by Mr. J. E. Howard, at the Watertown Arsenal, and it was shown by him at an early stage of the work that the kind of mortar was an exceedingly important factor. The inherent great strength of bricks can

† Watertown Arsenal Report, 1894, J. B. Johnson, Materials of Construction, p. 662.

* A Comparison Between the Rattler Test and the Sand-blast Test for Paving Bricks, Edw. Orton, Jr., Trans. Am. Ceramic Soc., 14, p. 180.

only be utilized by using a strong mortar. In ordinary brick masonry, therefore, even a fairly low-grade brick will possess ample strength. In places where strength is a real desideratum, cement or cement-lime mortar can be the only bonding material to be considered. Thus, Howard found columns, built of hard bricks and from 6 to 10 feet high, to show, with 1:3 lime mortar, a modulus of elasticity from 250,000 to 750,000 and an ultimate strength of from 750 to 1300 pounds per square inch. The piers were invariably found to split longitudinally, indicating that the tensile strength of brick is a very significant quality and that, hence, the transverse test is an important factor. Using Portland cement mortar (1:2) the modulus of elasticity of the pier was raised to 3,000,000 and the ultimate strength to 2500 pounds per square inch. Howard, at the Pittsburgh laboratory of the Bureau of Standards, also conducted tests of two very large brick columns, 4 feet square and 12 feet high, built with hard common brick from the Pittsburgh district. One was laid in 1:1 cement, the other in 1:3 lime mortar. The first showed an ultimate strength of 2917 pounds; the second, 757 pounds per square inch. The total compression of the lime-mortar pier was chiefly permanent set, while the cement-mortar column at 1000 pounds load was one-third permanent set and two-thirds elastic compression.

Recent Tests of Bureau of Standards. A considerable number of brick piers have recently been tested, at the Pittsburgh laboratory, by J. H. Griffith and J. G. Bragg,* in cooperation with the National Brick Manufacturers' Association. The size of the piers was about 30 x 30 inches, the height being 10 feet. Three kinds of bricks were used: vitrified, hard and soft. Three kinds of mortars were, likewise, employed—cement-sand (1:3), 85% cement, 15% hydrated lime to 3 of sand and lime mortar (1:6). Variations in bond were likewise made, viz., headers every other course, a header every fourth and every seventh course. All joints were made squeeze joints. The average thickness of the joints was 5/16 inch. The results of these tests are compiled in Table IV.

* The Clay Worker, March, 1915.

Table IV.

Kind of Brick	Kind of Mortar	Age	Bond	Maximum Load	Load in Lbs. per sq. inch
Soft (C)	Lime	4 mos.	1:1:5	129,000	126
Hard (B)	"	"	1:1:5	892,000	990
Vitrified (A)	"	"	1:1:5	1,280,000	1,360
Soft (C)	"	"	1:1:2	182,000	178
Hard (B)	"	"	1:1:2	804,000	890
Vitrified (A)	"	"	1:1:2	1,197,500	1,270
Soft (C)	"	"	1:0:1	215,000	210
Hard (B)	"	"	1:0:1	764,000	840
Vitrified (A)	"	"	1:0:1	1,360,000	1,450
Hard (B)	Cement-Lime	1 mo.	1:1:5	1,545,800	1,760
Hard (B)	" "	"	1:1:2	838,000	870
Hard (B)	" "	"	1:0:1	1,594,000	1,760
Vitrified (A)	" "	"	1:0:1	2,695,000	2,900
Soft (C)	Cement	1 mo.	1:1:5	660,000	650
Hard (B)	"	"	1:1:5	824,000	870
Vitrified (A)	"	"	1:1:5	2,697,000	2,900
Soft (C)	"	"	1:1:2	580,000	560
Hard (B)	"	"	1:1:2	1,834,000	2,070
Vitrified (A)	"	"	1:1:2	2,550,000	2,740
Soft (C)	"	"	1:0:1	524,000	510
Hard (B)	"	"	1:0:1	1,714,000	2,000
Vitrified (A)	"	"	1:0:1	2,520,000	2,710

Typical stress-strain curves of these piers are shown in the diagram of Fig. 4.

Conclusions from Pittsburgh Tests. From this work the authors draw the following conclusions: "The low strength of the lime mortars is attributed, in the main to insufficient ageing with a consequent lack of proper carbonation. The piers laid up with cement and cement lime mortar have about the same strength for any one grade of brick. The relative strength of piers built with vitrified, hard and soft bricks laid in cement mortar is approximately in the ratio of 5:3:1. The work so far indicates that the strength of the pier is largely independent of the course bonding, the real function of the bond being, it is believed, to maintain a certain integrity and monolithic action of the masonry against initial strains induced through setting and drying out of the mortar.

The slight increase of strength with the increase in distance between header courses is thought to be inconclusive. The mean moduli of the vitrified and hard bricks laid in cement mortar were 3,000,000 and 2,500,000, respectively, the cor-

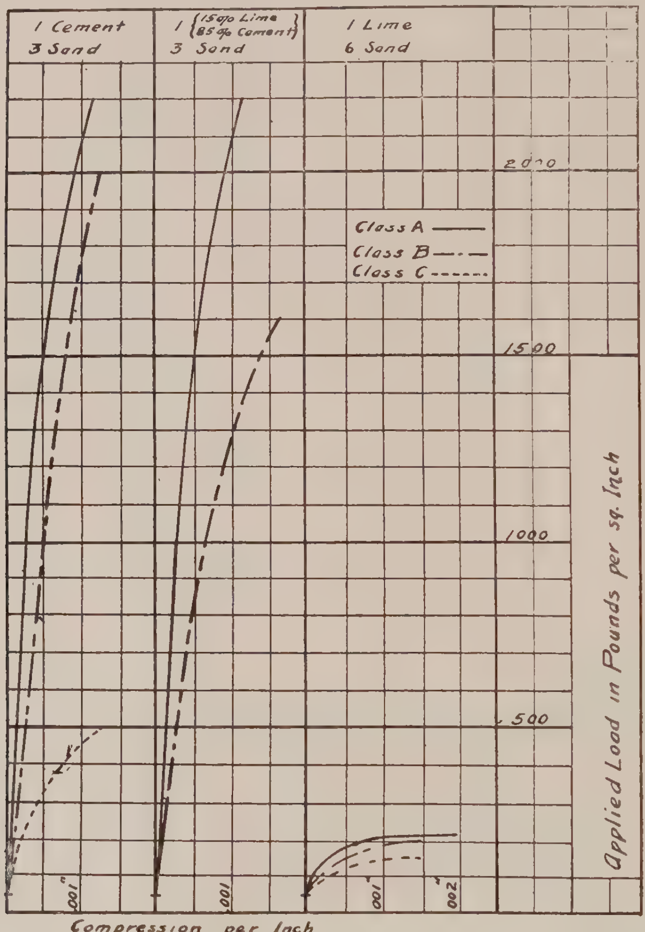

Fig. 4. Stress-Strain Curves of Tests on Brick Piers. Bureau of Standards.

responding figures for piers in the lime and cement mortar being somewhat lower. The elastic limit is approximately one-half the maximum load.

"The piers fail through a tendency to separate into vertical

laminae caused by transverse failure in the individual brick occurring from flexual action produced through unequal distribution of the vertical load over a cross section together with ineffective shearing and adhesive strength of the mortar. Uniformity in the brick dimensions is also a factor, to a lesser extent, in reducing unequal compression in the joints, as is the thickness of the joint itself.

"A high modulus of rupture is more desirable than superior crushing strength, as far as these be differentiated in their inter-relations. A higher modulus of rupture would be realized in practice either by increasing the thickness of the bricks or by laying them on edge so as to give their greatest flexual efficiency."

(l) Porosity and Resistance to Weathering.

Undoubtedly the most important quality of exposed structural stone products is their resistance to the weathering agencies, of which freezing and thawing are the most destructive. The properties which make certain materials weather resisting, while others succumb after a shorter or longer time, are not clearly understood. It is known, of course, that a vitrified brick, showing but little absorption, endures well, but when we are dealing with porous materials, the fact stands out that the one most absorbent, showing the greatest porosity, is not always the least resistant. Thus, porous bricks made by the soft-mud process may be decidedly superior in this respect to less porous bricks made by the stiff-mud process. It is evident that not only the volume of the pore space, but the mean diameter of the pores and their intercommunication are important factors. Bricks covered with an impervious coating, like a glaze or enamel, are subject to more severe stresses due to freezing and thawing than the same grade of material not so treated.

The only means of testing structural stone products as to their resistance to atmospheric agencies has been the freezing test. This test, while probably the best means of judging durability, is laborious and expensive, owing to the many repetitions. Other more indirect means of examination have been suggested, as for instance, the testing of specimens in the wet

and dry state. Any decided lowering of the strength due to the saturation with water is supposed to be an index of the weather resistance.

Accelerated tests have also been suggested as a substitute for the freezing test by making use of the volume change due to the crystallization of salts within the pores of the material. The most important method is Brard's process,* consisting in immersing the specimens in a boiling solution of sodium sulphate, Na_2SO_4. 10 H_2O, Glauber's salt, and exposing them to the air for a day in order that the salt may crystallize. This process is repeated. The solution of the decahydrate should be saturated when cold or at a temperature below 80° F. The loss in weight due to spalling may be computed. The average loss in weight by the sodium sulphate test is about six times that by the freezing test.‡ This test has also been used quite extensively by Prof. Bauschinger at the testing laboratory of the Royal Polytechnic School, Munich. This test was applied, in the Bureau of Standards laboratory at Pittsburgh, to three typical clays burned, in the shape of test pieces 4 x 4 x 2¾ inches, to different temperatures and subjected to four immersions (for 24 hours) in saturated sodium sulphate solution, followed each time by immediate drying at 110° C. Two series of specimens were prepared, one of which was fired to a somewhat higher kiln temperature than the other. The results were the same in each case and are compiled in Table V. From these data it will be observed that the test is sharp and decisive in breaking down the structure which has not reached its full strength. However, compared with the freezing test, it undoubtedly is too severe. It might be possible to modify it by using a lower concentration, say, a 10 per cent solution of sodium sulphate.

Elaborate procedures, as outlined by Hirschwald,† for the study of the weathering quality of stone are not readily applicable to clay products.

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* An. de Chem. et de Phys. Vol. 38, p. 160 (1828), later modified by d'Hericart and de Thury. Trans. Am. Soc. C. E., Vol. 33, p. 246.

‡ Trans. Am. Soc. C. E., Vol. 33, p. 253.

† Die Prüfung der natürlichen Bausteine auf ihre Wetterbeständigkeit, Berlin, 1908.

Table V.

	Burning Temp.	Absorption %	Absorption % (check)	Remarks
Surface clay	900	21.7	19.4	Cracked badly on first treatment.
" "	950	21.2	19.4	Failed on second treatment.
" "	1000	21.2	18.0	Cracks became prominent on second treatment.
" "	1050	19.1	14.8	Failed on third.
" "	1100	6.3	1.3	Not affected.
" "	1150	0.6	Overburned	Not affected.
Shale	900	19.6	17.7	Many fine cracks after first treatment.
"	950	18.4	17.3	Failed on second treatment.
"	1000	14.2	12.0	Cracks increasing through first three treatments; failed on fourth.
"	1050	9.3	8.1	Not affected.
"	1100	2.8	3.0	Not affected.
"	1150	2.3	0.4	Not affected.
"	1200	2.7	1.3	Not affected.
No. 2 fire clay	900	15.8	15.3	Edges chipped after first treatment; failed on second.
" " "	950	15.0	15.1	Edges chipped after second treatment.
" " "	1000	13.5	11.6	Failed on third.
" " "	1050	10.5	9.6	Cracked badly on fourth treatment.
" " "	1100	8.2	8.3	Not affected.
" " "	1150	5.8	5.8	Not affected.
" " "	1200	3.5	3.0	Not affected.

The remarkable resistance of well made clay brick to destruction by weathering is amply illustrated by the mediaeval brick architecture of Central Europe, not to mention the brick architecture of Babylonia and Assyria.

(m) Resistance to Sudden Heating and Cooling.

In the large problem of fire protection, the resistance to sudden heating and cooling is an important factor. It is to be expected that clay products should stand up well under these conditions, having already gone through the ordeal of fire in their manufacture and, hence, containing no volatile constituents. Their resistance to such conditions must be considered a function of their porosity. The more porous the bricks, the better they should withstand sudden temperature changes. A

certain degree of homogeneity, absence of larger lumps of foreign material, is also a necessary condition.

In large panel tests made at the Underwriters' Laboratories and reported by Mr. Richard L. Humphrey* it was found that bricks withstood the tests better than any other material. Lime knots seemed to be responsible for damage done to the brick. Dense vitrified bricks probably do not stand up so well under severe conditions as the more porous brick. Comparison with natural stones, in these fire tests, showed that bricks were superior to them. There is undoubtedly need for further work along these lines.

Actual fire experience seems to give undoubted proof of the excellent fire resisting qualities of clay brick.

With reference to clay building bricks as a structural material, it might be said that they undoubtedly afford one of the very safest products available, as usually their strength is ample for the purposes required and any inferior quality is at once detected by the most inexpert by simple inspection. Bricks have stood the test of time and are not likely to lose their importance as a building material in the future.

PAVING BRICK.

The extensive use of vitrified bricks for the paving of streets and roads has led to the adoption of rigid requirements for this class of materials. The test used for this purpose is the so-called rattler test, a standardized tumbling barrel in which ten blocks are subjected to abrasion, both from each other and from cast-iron spheres, the standing of the samples depending upon the loss in weight suffered after one hour's run (1800 revolutions). After considerable experimental work on the part of the National Paving Brick Manufacturers' Association and Prof. Edw. Orton, of the Ohio State University, standard specifications for the rattler and for the procedure of the test have been recommended.†

The following scale of average loss has been suggested for different requirements:

* U. S. Geological Survey, Bull. 370.

† Proc. Am. Soc. for Testing Materials, 23, p. 289.

	General average loss, percent	Maximum permissible loss, percent
For bricks suitable for heavy traffic	22	24
For medium traffic	24	26
For light traffic	26	28

Owing to the severity of this test, the absorption and other factors are no longer taken into account for paving brick.

In the use of paving bricks, the proper laying of the blocks is of paramount importance. Unless a good concrete foundation is prepared, the sand cushion applied and rolled uniformly, the grouting thoroughly done and proper longitudinal expansion joints allowed, even the very best blocks will not result in a pavement of the highest type. It is for this reason that explicit specifications have been worked out by the National Paving Brick Manufacturers' Association with great care and persistently advocated. Given proper conditions of laying, brick streets of magnificent wearing quality will be obtained, such as has been realized in the many miles of brick-paved streets in Cleveland, Ohio, and other cities.

HOLLOW BUILDING TILE AND FIRE-PROOFING.

(a) Hollow Tile in General.

The use, in the form of hollow tile, of larger units than the brick is growing rapidly in the favor of the building public. Of these, a considerable number of sizes are made, without webs or with webs, in one or both directions. Interlocking tiles have also come into use to a considerable extent. Two classes of tile may be distinguished, one in which the tiles are placed vertically and one in which they are set in the horizontal position.

These tiles are being used both for bearing and non-bearing walls. For the former purpose, only hard-burned tile can be considered, while soft material is well suited for partition walls. Hollow tiles are particularly useful for enclosure walls of steel or concrete skeleton structures and for stucco work.

The crushing strength of single tiles, hard burned, may vary from 4000 to 10,000 pounds per square inch of effective area, and the modulus of elasticity from 3,000,000 to 5,000,000. Quoting from specifications proposed by the Hollow Building Tile Manufacturers' Association, "All tiles shall be true,

straight and free from objectionable cracks, shall be well burned and of sufficient hardness not to absorb more than 10 per cent of their weight of water in a 48-hour absorption test, and shall have an ultimate crushing strength of not less than 3000 pounds per square inch of net web area tested * * *''. ''No tile in any wall shall be loaded, including all live and dead load, so as to exceed 200 pounds per square inch of their bearing members. No hollow tile shall contain any void whose cross section dimension, measured at right angles to the web, is more than 4 inches, and the vertical webs and shells of all tiles used for all bearing walls shall have a thickness of not less than 15 per cent of the measurements across the void enclosed by such shells or webs.'' * * * ''For a one-story building the thickness of the wall shall not be less than 6 inches; for a two-story building 8 and 6 inches; for a three-story structure 8, and beyond the second story 6 inches; for a four-story building, 12 inches for the first and second story, 8 inches for the third and 6 inches for the fourth story''.

(b) Strength of Hollow Tile Wall Sections.

It is evident that, in construction of this kind, the most important information is obtained from the testing of built-up wall sections and piers. A considerable amount of this work has been done by the National Fireproofing Company and other concerns. In Table VI some results of such work are compiled. For most of these results the writer is indebted to Mr. E. V. Johnson of the National Fireproofing Company. From these tests it appears that the quality of the mortar, as was the case with bricks, is a most important function. In the examples where lime-cement mortars were used, the addition of cement was quite small. The laying of tiles in the vertical position is greatly facilitated by the use of strips of wire screens, which also impart to the wall a very high degree of rigidity.

The stability of tile walls is very marked indeed and may exceed that of brick walls of equivalent dimensions. In regard to the fire resisting quality of tiles, the same conditions prevail which have been mentioned under the head of bricks. Where this quality is to be emphasized, a certain degree of porosity is essential. This applies especially to fire-proofing used for the protection of steel columns and girders. For this

purpose products made from No. 2 fire clays are especially suitable, as they are of a porous nature and, at the same time, of good refractoriness, able to withstand very high temperatures. Very low-fusing clays, vitrifying easily, should not be used for fire-proofing purposes.

The subject of tile floors and arches and of combined tile and concrete construction is a large one and the testing data available are not extensive. The large and successful use of tiles for this purpose seems to speak well for the use of this material.

The excellent thermal non-conducting quality of hollow tiles makes them particularly desirable for residence construction. This statement applies, of course, only to atmospheric temperatures. A hollow-tile wall would not be desirable for furnace walls, owing to rapid increase of heat loss by radiation through the air spaces, with increase of temperature.

ARCHITECTURAL TERRA COTTA.

What has been said of the strength of burnt clay in the form of bricks and tiles applies also to terra cotta, which might be said to be midway between these two kinds of product. The data available in regard to this subject are quite meagre. The subject of the strength of terra cotta is not very important, inasmuch as this material is used principally for facing purposes, although it is nearly always well burned and of excellent strength. It is generally made from No. 2 fire clays, and, with an absorption of 10 per cent, shows an average crushing strength of not less than 4000 pounds per square inch. The resistance to weathering of the glazes and enamels is to be considered, and it is essential that the covering slips and glazes be firmly united to the body of the ware and that the enamels be sufficiently resistant to make them sufficiently weatherproof. Freezing tests of terra cotta would be very desirable and an acid test would gauge the resistance of enamels to atmospheric influences.

SEWER PIPE AND DRAIN TILE.

(a) Resistance to External Pressure.

Sewer pipe, to serve its purpose most successfully, should be composed of a strong, practically vitrified body showing

about 4 per cent absorption and should be well glazed. Pipe is invariably subjected to external pressure, due to the weight of the material above it, which in the case of wet clay at a depth of 10 feet or more amounts to 69% of the total superimposed weight, according to the conclusions of F. A. Barbour*. The heaviest loads to which sewer pipe may be subjected are produced by trucks and steam rollers. Mr. Harrison P. Eddy† estimates the pressure which may thus be produced in the case of a steam roller at about 6500 pounds per linear foot of trench. This investigator calculates that about 25% of this load is applied over the full width of the trench at a depth of nine feet.

F. A. Barbour proposed a formula for the required thickness of sewer pipe as follows:

$$t=\frac{1.65}{}\sqrt{\frac{pd}{c}}$$

where t = thickness of shell in inches,
p = pressure per linear foot in pounds,
d = internal diameter of pipe in inches,
c = constant = 33000.

It was assumed by him that the estimated tensile strength of clay is 900 pounds, and the compressive strength 6500 pounds per square inch. He recommended that standard pipe be designed for a breaking load of 3000 pounds and double-strength pipe for a breaking load of 4500 pounds per linear foot. Similar results are obtained by the use of the general formula for pipe by Talbot.‡

$$t=\tfrac{1}{4}\sqrt{\frac{6wd'}{f}}$$

where t = thickness of shell in inches,
w = total distributed vertical load in inches,
d' = mean diameter in inches = internal diameter + thickness of shell,
f = ultimate tensile strength in pounds per square inch in outer fibre,
using the values employed by Barbour.

* Jour. Assn. Eng. Soc., 19, p. 193.

† Report of Committee C-4, The American Soc. for Testing Materials.

‡ Jour. Assn. Eng. Soc., Dec., 1897.

In Tables VII and VIII, results of crushing strength determinations of sewer pipe are given. The first of these tables represents results obtained by F. A. Barbour*, the second, determinations by the Testing Laboratory of the Bureau of Sewers**, Brooklyn, N. Y. In the latter, the tests were made with pipe in a box containing sand. The pressure was applied through a wooden knife edge.

Table VII.

Crushing Strength of Vitrified Clay Pipe.

Standard.

Nominal size of pipe Inches	Number of pieces broken	Thickness of shell Inches	Distributed vertical load per linear foot of pipe Pounds
6	9	.72	2589
6	13	.67	2638
8	6	.78	1662
8	5	.822	2962
8	11	.821	2843
10	11	.832	3445
12	5	1.02	1904
12	6	1.05	3134
12	6	1.00	3319
15	6	1.23	3435
15	4	1.13	2979
18	6	1.43	2771
18	3	1.26	2504
18	11	1.18	2768
20	4	1.29	2754
20	12	1.32	2414
24	6	1.48	2334
24	5	1.46	2938
24	10	1.48	2377
	Double Strength.		
12	6	1.26	3916
15	5	1.45	4710
15	6	1.36	4414
18	5	1.56	4077
18	6	1.52	4216
20	6	1.72	4119
20	4	1.76	3117
24	3	2.02	4334

* Jour. Ass'n. Eng. Soc., 19, Dec. 1897.

** Munic. Jour. and Eng'r, 28, Feb. 1910.

Table VIII.

Crushing Strength of Vitrified Clay Pipe.

Nominal size pipe inches	Total number tested	Average pressure lbs. per lin. ft. of pipe	Highest result lbs. per lin. ft. of pipe	Lowest result lbs. per lin. ft. of pipe	Bureau Standard required Pressure Lbs. per lin. foot
6	170	1537	2333	1033	1000
12	245	1542	2900	933	1150
15	72	1935	2800	1300	1300
18	25	2389	3100	1734	1450
24	17	2825	3800	2200	2000
30*	2	3260	3280	3240	....
30†	2	3160	3240	3080	....

With reference to the thickness of pipe, Fruehling gives as the German practice the relation $t = \frac{d}{20} + 0.39$ inches,

where t = thickness of pipe shell in inches,

d = internal diameter of pipe in inches.

Pipes from 9 inches in diameter upwards are usually made in two thicknesses, 1/10 and 1/12 of their diameter.

(b) Resistance of Pipe to Internal Pressure.

Work has been done along this line by Professor M. A. Howe* at the Rose Polytechnic Institute and Burchartz and Stock at the Royal Testing Laboratory, Gross-Lichterfelde**. In the work by Howe, the averages for the different products varied from 265.6 to 1081.8 pounds per square inch. The general average of all the results is 600.4 pounds per square inch. The ultimate average tangential stress in the German results varied from 490 to 1120 pounds per square inch, and the total average internal water pressure from 7 to 26.1 atmospheres. The average water pressure resulting in failure was 13.9 atmospheres; the ultimate average stress, 850 pounds per square inch. The results of Burchartz and Stock are given in Table IX.

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* Jour. Ass'n. Eng. Soc., June, 1891.

** Eng. Rec., 54, p. 193.

Table IX.

Summary of the mean minimal, maximal and average values for resistance against internal pressure with vitrified clay pipe.

Average internal diameter (inches)	Average thickness of shell (inches)	Number of individual tests	Internal water pressure, in atmospheres. Average Minimal	Average Maximal	Total Average	Ultimate average tangential stress (Lbs. sq. in.)
2	0.72	3	24.0	28.0	26.1	490
3	0.80	3	15.0	18.4	16.3	490
4	0.68	11	9.2	25.2	20.1	830
6	0.80	23	8.3	24.5	17.2	850
8	0.92	13	14.6	24.2	17.9	1120
10	0.96	2	6.0	8.0	7.0	600
12	1.04	19	6.7	16.9	11.9	920
16	1.12	2	9.4	9.7	9.6	910
18	1.28	8	7.1	12.0	9.3	920
20	1.44	4	9.0	12.2	10.6	1040
24	1.72	7	6.3	8.6	7.2	720
28	1.88	4	7.1	10.2	8.7	910
32	1.96	14	5.4	9.0	7.7	1120
Average					13.9	850

With reference to the resistance of vitrified well-glazed sewer pipe to the corroding influence of sewage, there can be no question but that such a surface is ideal from this standpoint. No other material can be expected to show such inertness to chemical agents and electrolysis. The question of proper joints is an important one and perhaps the best treatment of this point has not yet been developed.

(c) Drain Tile.

An exhaustive study of the strength of clay drain pipe has been made by Committee C-4 of the American Society for Testing Materials. The required average supporting strengths of tile, tested according to methods outlined by the Committee*, are compiled in Table X.

* Year Book, Am. Soc. Test. Materials. 1914, p. 323.

Table X.

	Pounds per linear foot.		
Diameter	Class No. 1 B	Class No. 1 A	Class No. 1 Extra A
10	600	--------	--------
12	700	900	1000
14	800	1000	1200
16	900	1200	1500
18	1000	1300	1700
20	1100	1400	2100
22	1100	1550	2300
24	1200	1700	2500
26	--------	1800	2600
28	--------	1900	2800
30	--------	2000	3000
32	--------	2050	3200
34	--------	2150	3300
36	--------	2250	3500

No. 1 B tile are intended to be suitable for supporting the load in the worst material in a trench having a grade line 5 feet deep. Class No. 1 A tiles shall represent good materials and are intended to be suitable for supporting the load in a trench having a grade line 7 feet deep.

No. 1, Extra A tiles shall be very good and vitrified and shall be able to support the load in the worst material in a 10-foot trench.

REFRACTORIES.

Fire clay refractories, according to their resistance to high temperatures, are divided into No. 1, No. 2 and No. 3 materials. The first class of refractories, when tested in a suitable furnace, should be able to withstand a temperature of not less than that corresponding to pyrometric cone No. 31 (approximately 3065° F.); the second class, cone 28 (about 2975° F.); and the third, cone 26 (approximately 2912° F.). It is extremely important that fire brick should be burned, in manufacture, to a high temperature, in order that they should remain as constant in volume as possible when built up into furnace walls.

A test bringing out this behavior is very important. It consists in heating up full-size bricks, in a properly constructed furnace, to the temperatures corresponding to pyrometric cones Nos. 14, 12 and 8, respectively, i. e., 2570°, 2462° and 2282° F., for the three classes of products and holding the temperature for six hours. The linear contraction or expansion (in the case of silicious materials) should not be more than 2 per cent.

In many constructions the load-carrying ability of fire-bricks and other shapes is an important factor. This applies especially in gas benches, where loads as high as 25 pounds per square inch may be reached. Hence a load test bringing out the behavior of clay refractories seems essential. In the Pittsburgh Laboratory of the Bureau of Standards, such a test has been worked out, and consists in applying a load of 50 pounds per square inch at a maximum temperature of 1350° (2462° F.). This temperature is reached in five hours and is maintained for one hour. The load is applied by means of a weighted beam, the pressure being transmitted through an opening in the crown of a gas-fired furnace by means of a very refractory column. The bricks are tested on end. High-grade clay bricks should show a contraction of not more than 7 per cent of the original height and no evidence of flow, deformation or cracking. It is quite possible that high-grade bricks, well grogged with properly-sized calcined material, can be made which show practically no contraction, though only a few brands of this kind are being manufactured at the present time.

The load test* may, in fact, be considered a modified fusion test. Frequently the fusion point determination is misleading, owing to the fact that clays do not possess a melting point in the sense of metals and minerals. Owing to its high viscosity, a specimen of fire brick may stand up under its own weight to quite a high temperature, although it would give way at a fairly low temperature under the conditions of pressure prevailing in a furnace crown or gas bench. The load test, therefore, overcomes this condition and tends to rate the material at its true value.

Chemical analysis frequently is of service in estimating the value of a refractory. Two kinds of clay fire-brick may be con-

* Bureau of Standards Technologic Paper No. 7.

sidered to exist, those approaching the composition of the pure clay substance, 53.8% silica and 46.2% alumina, in the burned condition, and those of a more silicious nature, with as high as 85% silica. The latter approach the so-called silica brick in their properties, being less refractory, showing excellent standing up qualities in the load test, expanding instead of contracting in use, and being more sensitive to sudden temperature changes. Fluxes like iron oxide, potash, soda, lime and magnesia are more effective in lowering the softening point of silicious clays than of the aluminous type. High-grade clays should not contain more than 5 per cent of the above fluxes.

For conditions involving rapid temperature changes, porous high-grade clay brick, containing a large amount of calcined material, are more satisfactory than silicious materials.

Other functions are frequently of considerable importance, such as resistance to the attack of slags, glasses and reducing gases. In such cases proper sizing of the calcine, dense pressing and hard burning are essential.* In addition, close mortar joints are important. Tests can be made for the estimation of this quality by making cubes of the fibre-clay body containing a recess in which a quantity of the slag is put. By heating the specimen beyond the melting point of the slag, the degree to which the structure withstands the attack may be determined.

Thermal Conductivity.

The thermal conductivity of fire-brick is a direct function of the porosity. Thus, a dense brick showed at 1400° F. a heat flow of 124 B. t. u. through one square foot area, one inch thick, per 24 hours, and for a temperature difference of 1° F. A somewhat more porous brick, under the same conditions, conducted 87 and a kieselguhr brick, extremely porous, 22 B. t. u. The thermal conductivity of fire-brick varies also with the temperature, as is shown in recent work by Heyn† which is compiled in Table XI. Here the value of K represents the heat flow, in calories per second, through 1 sq. cm. and 1 cm. thickness, per °C, temperature difference. The figures are of the order $K = 0.001 = 100$.

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* Gilbert Rigg, Jour. Ind. and Eng. Chemistry, 5, No. 7.

† Stahl und Eisen, 34, p. 832. Also Mitteilungen aus dem Kgl. Materialspruefungsamt, Berlin, 1914, Nos. 2 and 3.

Table XI.

Kind of fire-brick	K at the temperatures, °C. 200	400	600	800	1000	1100	1200
High Clay	145	180	220	240	260	270	......
More silicious	115	140	165	190	215	......	......
Silicious	90	110	125	130	130	......	......
Silicious	210	245	270	270	270	270	270
Silica brick	135	160	170	170	180	210	240
Magnesite	110	115	120	130	140	140	......

It seems that reheating changes the conductivity of fire-brick as the structure is altered.

The specific heat of fire-bricks likewise changes with the temperature. This relation is expressed by Mellor* in the equation:

$$S = 0.193 + 0.00006\ t,$$

where S = mean specific heat and
t = temperature, °C.

The true specific gravity of the fire-bricks varies from 2.622 to 2.755, as determined upon 28 specimens of American materials.†

The coefficient of expansion of fire clays, again, is a function of the temperature, as well as the number of reheatings to which the specimens have been subjected. The available data are very meager. P. A. Boeck†† thus found the coefficients for ball clay burnt to cone 12 to be as follows:

	First run	Second run
Between 24° to 100° C.	1316×10^{-8}	1310×10^{-8}
Between 100° to 200° C.	1710×10^{-8}	1610×10^{-8}
Between 200° to 700° C.	558×10^{-8}	580×10^{-8}
Between 200° to 900° C.	....	567×10^{-8}

These relations also show discontinuities, indicating thermal reactions. In the case of a fire clay the mean coefficient of expansion (referred to 1° C.) was found to be 0.0000097.

* Trans. English Ceramic Soc., 12, part II, p. 279.

† Bureau of Standards, Technologic Paper No. 7.

†† Trans. Am. Ceramic Soc., 14, p. 470.

FLOOR TILES, WALL TILES, ROOFING TILES, AND ENAMELED BRICK.

(a) Floor Tiles.

Two kinds of floor tiles are commonly used, the thoroughly vitrified, non-absorbent variety, in various colors, and the hard usually dark red "quarries", which, though well vitrified, are not completely impervious.

Where sanitary conditions are the main consideration, only non-absorbent tiles can be used. In such materials the absorption should not be more than 0.05 per cent. The so-called ink test may be misleading; hence, it is best to rely upon the absorption. Other tests may be applied, as the transverse and the ball impact test. These have not been sufficiently standardized to make possible the fixing of limits. Some work along these lines has been reported by F. B. O'Connor.* In these tests the average modulus of rupture was found to vary from 7422 to 3262. The highest values are shown by the porcelain tiles, either white or colored by means of various oxides. The sandblast or some sort of abrasion test might likewise be of service. The ball impact test can be made to give interesting data.

(b) Wall Tiles.

The principal consideration in glazed wall tiles is the freedom of the glazed surface from crazing or a tendency to shatter. By boiling the tiles in a saturated solution of common salt for three hours, the crazing difficulty is emphasized, and checking of the glaze occurs if a strain between body and surface exists.

(c) Roofing Tiles.

Specifications for this material do not exist, but it is evident that good tiles should be able to stand up in the freezing test. Absorption limits cannot be given, as there are on the market both well-vitrified and rather porous tiles which seem to have stood up quite satisfactorily. On general principles, the hard-burned variety is to be preferred.

(d) Glazed and Enameled Brick.

High grade bricks of this kind should possess a hard glaze or enamel, matured not lower than pyrometric cone No. 3. Soft lead glazes are not satisfactory for exposure to the atmos-

* Trans. Am. Cer. Soc., 15, p. 233.

phere. The clay should be burned to an absorption of not more than 8 per cent. The bricks, on being subjected to the freezing test, should show no evidence of disintegration. The glaze should be practically insoluble in hot 10 per cent hydrochloric acid solution. In regard to the modern demand for more and more aseptic conditions, the glazed surfaces of this kind of product offer a satisfactory solution.

SANITARY WARE.

The plumbing fixtures included in this class of ware consist of two classes, one made of a vitrified porcelain body and another of a porous fire-clay body.

It is evident that the former class represents a more valuable product, since chipping would not expose an absorbent fracture. No specific requirements exist for these products, excepting that the vitreous body must show practically no absorption, and that the glaze should be free from crazing and similar defects.

Within the scope of this paper, all clay products entering into engineering could not be discussed. Such materials as electrical insulators have not been considered, as this was not possible without unduly enlarging the volume of this contribution.

Clay products have not, in the past, received the attention from engineers and investigators which their importance deserves. It would be well worth while if more consideration were given this subject in the curriculum of the architectural and engineering departments of our colleges. This would assist greatly in enlarging the incomplete store of data now available.

DISCUSSION

Mr. C. H. Kromer,* Assoc. M. Am. Soc. C. E., said that brick joints which are thicker than one-half inch lose in strength. If the mortar joint is stronger than the brick, it makes no difference. Mortar is the chief factor in the strength of brick piers. Thick joints are used for artistic effect. It would be interesting to know how the strength of brick piers depends upon joints. Mr. Kromer.

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Paper No. 101

CONCRETE AGGREGATES.

By

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Concrete has taken a recognized place in engineering and architectural construction. This has come about to a large extent through the uniformity and low cost of the manufacture of Portland cement. The other principal ingredient of concrete, the aggregate, which is really of equal importance to the cement, has not been thoroughly standardized. The principal reason for this is because, while cement is manufactured under factory conditions and subjected to routine chemical and physical tests, the aggregate, which constitutes the largest bulk of the concrete, is usually taken from the immediate vicinity of the job to avoid the cost of long transportation. The engineer may select and accept cement knowing nothing whatever of its characteristics and manufacture, but only that it comes from a recognized mill and passes routine tests. To select his aggregate, that is, his sand and stone, and be sure that he is getting a safe and economical material, he must thoroughly understand the principles of proportioning, must be able to choose his materials, and to adapt them to each portion of his structure.

It is the purpose in this paper not to present a theoretical treatise, in fact, the prescribed length would prohibit this, but to treat in a practical manner the fundamental principles involved, so as to give to the reader, especially to one who is not a specialist in concrete construction, information which will be of practical value.

The subject will be taken up along the following lines:

Historical
Aggregates of the United States

Economy in selection of aggregate
Requirements for concrete
Effect of the aggregate upon the quality and strength of concrete and mortar
Tests of quality
Tests for acceptance

Attention is called to the table of commercial tests and the conclusion that sand for concrete should always be tested.

HISTORICAL.

The first presentation of scientific investigations of the characteristics of aggregate dates back only as far as 1892. Mr. R. Féret in Annales des Ponts et Chaussées gave a masterly presentation of a series of tests illustrating different properties of aggregates in mortar and the laws governing their use. This was followed, in 1896, by a second paper by the same author, in the Annales, and in 1897 by an extended paper in the Bulletin de la Société d'Encouragement pour l'Industrie Nationale. Previous to Mr. Féret's paper in 1890, Mr. P. Alexandre presented a paper in the Annales on "Hydraulic Mortars", but his tests are less scientific than Mr. Féret's. In the United States, scientific series of tests with different sands and aggregates were made at St. Mary's Falls under the direction of Mr. Louis C. Sabin, and are recorded in the Reports of the Chief of Engineers, U. S. A., 1895 and 1896. In a paper before the American Society for Testing Materials,* Professor M. O. Withey presents results of mortar tests with different sands, which are confirmatory of certain of the conclusions of Mr. Féret. The author of the present paper, also, has checked Féret's results in many particulars.

In the line of concrete mixtures, that is, mixtures with coarse and fine aggregate, still fewer investigations have been made that really treat the matter in a thorough manner. In 1907 Mr. William B. Fuller and the author presented before the American Society of Civil Engineers a paper on "Laws of Proportioning Concrete", based on an extensive series of tests at Jerome Park Reservoir, New York, which gives the results of tests and certain laws deduced from them.

* Proc. American Society for Testing Materials, Vol. XIII, 1913, p. 834.

More recently—under the auspices of the Committee on Specifications and Methods of Tests of Concrete Materials of the American Concrete Institute and a similar committee acting jointly with this, known as the Committee C-9 on Standard Tests of Concrete and Concrete Aggregates, of the American Society for Testing Materials—series of tests have been made, and are still in progress, for the clearer formulation of laws of concrete mixtures.

AGGREGATES OF THE UNITED STATES.

Fortunately for the development of concrete construction, aggregates suitable for use as a concrete material are available in nearly all sections of the country. In certain localities, however, the sand obtainable in natural deposits is of very inferior quality, and it is frequently not only safer but actually more economical to transport sand or fine crushed stone from a considerable distance than to use inferior material with enough cement added to make it acceptable. The trouble usually found is in obtaining a sand that is coarse enough to give the requisite strength to the concrete. More often than is apt to be recognized, also, cases are occurring where the strength is greatly affected by impurities in the natural bank. New York, which is using in its public works and public service structures immense quantities of concrete, is particularly favored by having at hand, on Long Island, immense deposits of most excellent sand and gravel, which are screened by large plants to commercial sizes. New England, over which are distributed glacial deposits, is very rich in sands and gravels, but the character of the material is extremely varied and the author has found case after case of sand, coarse and of apparently good quality, which tests have proved to be unfit for use. A number of these are illustrated in the table of tests below. In certain sections of the Middle States there are excellent quartz sands, but care must be exercised to prevent the acceptance of too fine sand. In northern New York, certain sections of Canada, and other localities in this general latitude are many sands composed largely of limestone grains. In the South, fine earths predominate; and in many sections it is difficult to find a sand which is sufficiently coarse and free from clay to be suitable for concrete construction. In the West,

owing to the diversified character of the country, all qualities of material are to be found.

Crushed stone, that is, the screenings from a crusher plant, is used only to a limited extent for fine aggregate, although material from this source frequently makes excellent mortar, equal to or superior to natural sands. Screenings from a soft stone are unsuitable because giving too large a proportion of dust. For the Kensico dam for New York City, now under construction, considerable quantities of screenings were used as fine aggregate, either alone or mixed half and half with sand. A test of these screenings is given in the following table.

Representative sands and crushed stone screenings are illustrated by full size photographs in Fig. 1.* These include five samples of natural bank sand, all of them used extensively for concrete, although one, that from Tennessee, is of poor quality because of its fineness. The high strength of the crushed stone screenings, notwithstanding the large amount of dust, is noticeable.

The following table gives the tensile strength and mechanical analyses of the sands shown in the photographs, Fig. 1.

For coarse aggregate, gravel is almost always suitable, if it runs up to the proper size and is clean or can be washed.

Bank gravel is sometimes used for the aggregate without screening, but since the best proportion of sand to stone seldom occurs in nature, it is almost always best to screen and remix.

As indicated below, almost any kind of crushed stone is suitable if it is not too soft.

Slag, if acid slag properly air-cooled, is being accepted as a coarse aggregate for concrete.

Cinders are restricted to use as fire-proofing and for floor slabs of short, say up to 8-foot, span.

ECONOMY IN SELECTION OF AGGREGATES.†

The most expensive ingredient of concrete is the cement. Although this occupies usually only from 10% to 15% of the

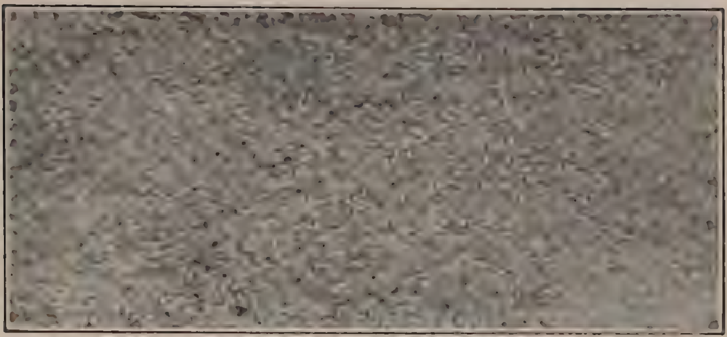
* Photographs especially taken for this paper by courtesy of Stone and Webster Engineering Corporation.

† Information illustrating the effect upon the cost of aggregates under different conditions is given in "Concrete Costs" by Taylor and Thompson, pp. 158 to 164.

TESTS OF SANDS SHOWN IN PHOTOGRAPHS.

		Tensile Strength of 1:3 Mortar and Ratio of Strength to 1:3 Standard Sand Mortar						Mechanical Analysis % passing sieves No.				
		3 Days		7 Days		28 Days						
		Lb. per Sq. In.	Ratio %	Lb. per Sq. In.	Ratio %	Lb. per Sq. In.	Ratio %	¼″	12	40	100	200
(1)	Standard Ottawa Sand	175	100	230	100	315	100	Passed No. 20 and retained No. 30				
(2)	Wellesley Sand, Mass.	157	91	230	100	322	102	100	77	29	4.7	2.0
(3)	Plum Island Sand, Mass.	140	81	207	90	304	96	100	80	8.5	3.2	0.6
(4)	Cowe Bay Sand, N. Y.	261	151	258	112	311	98	100	86	31	2.07	1.58
(5)	Waupaca Sand, Wis.*	230	134	322	140	443	140	100	87	25	0.8	0.16
(6)	Jersey Gravel Screenings, N. J.	86	48	120	52	195	61	100	85	49	10.1	3.1
(7)	Tennessee River Sand	62	36	103	45	184	58	100	97	63	2.3	0.8
(8)	Rockport Granite Screenings, Mass.	189	108	276	120	440	138	100	70	35	18.0	11.7
(9)	Winchester Trap Screenings, Mass.	161	92	296	129	465	146	100	43	23	10.9	5.7
(10)	Chicago Limestone	242	138	326	142	522	163	100	60	29	18.0	13.0

* Exceptionally clean.

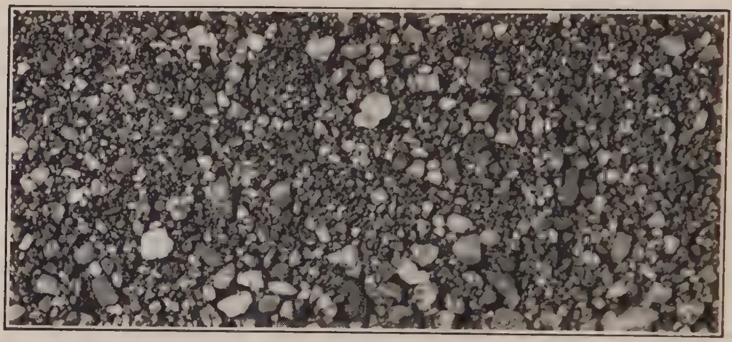


Standard Ottawa Sand.

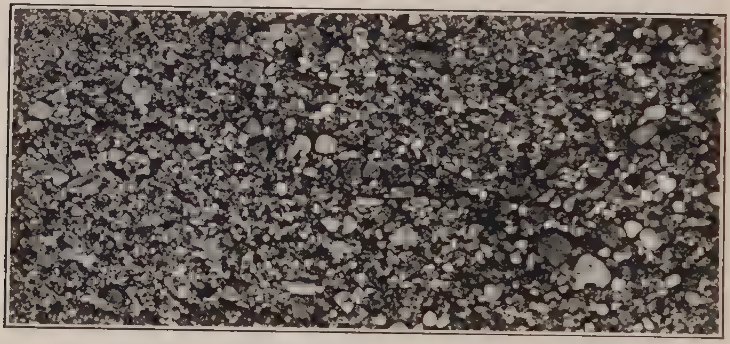
Wellesley Sand Mass

Plum Island Sand, Mass.

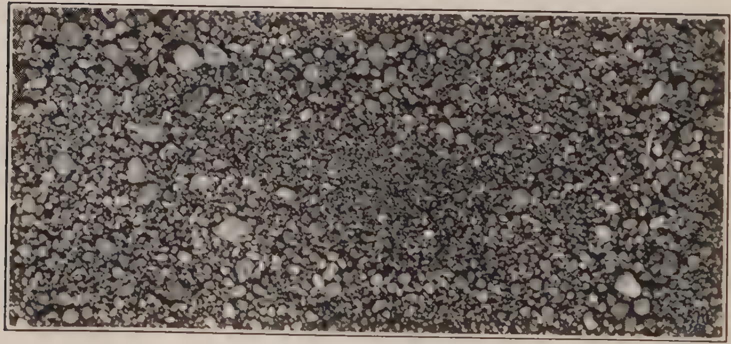
Fig. 1. Representative Sands and Crushed Stone Screenings.



Cowe Bay Sand, N. Y.



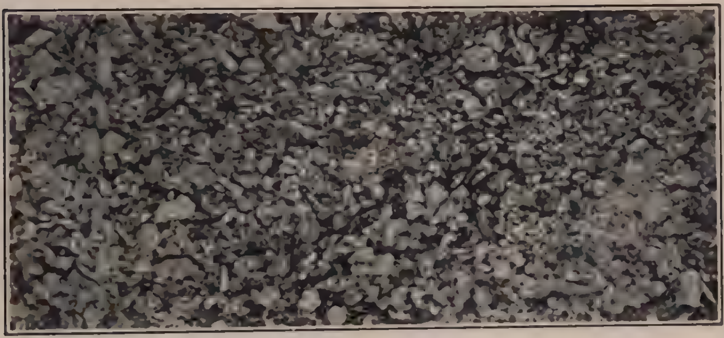
Waupaca Sand, Wis.



Jersey Gravel Screenings, N. J.

Fig. 1 (Cont.) Representative Sands and Crushed Stone Screenings.

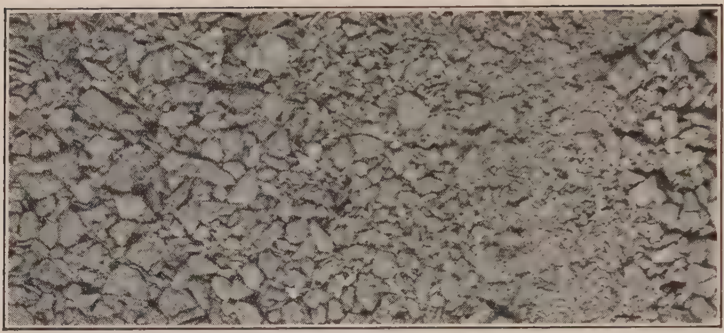
Tennessee River Sand.



Rockport Granite Screenings, Mass.

Winchester Trap Screenings, Mass.

Fig. 1 (Cont.) Representative Sands and Crushed Stone Screenings.



Chicago Limestone Screenings, Ill.

Fig. 1 (Cont.) Representative Sands and Crushed Stone Screenings.

mass, its cost is apt to be as much as one third of the cost of all the materials. It is evident, therefore, if we can reduce the quantity of cement, that there will be a great reduction in the cost of the concrete. The field has been investigated up to this time to only a small extent.

In the manufacture of concrete in large masses, appreciable economy can be effected by careful advance study and test. For example, at Little Falls, N. J., Mr. William B. Fuller was able to produce by proper proportioning of the concrete a change in proportions from 1:2:4 to 1:3:7, thus reducing the quantity of the cement from 1.57 barrels per cubic yard to 1.01 barrels, and effecting a saving, allowing for extra cost of labor, of about 75c per cubic yard.†

For commercial manufacture of artificial stone, in only a few cases is proper attention paid to the selection and grading of the aggregates. Not long ago the author of this paper, by a series of commercial tests, was able to attain, with proportions of one part cement to 7 parts aggregate, stone as strong as a mixture of one part cement to 4 parts aggregate, formerly used, with a consequent saving of about 60c per cubic yard of stone. This was accomplished by increasing the maximum size of the aggregate from ⅛ inch to ⅜ inch and following out established principles of mixing aggregate to obtain the greatest density.

† See Taylor and Thompson's "Concrete, Plain and Reinforced", 2nd Ed., p. 183.

REQUIREMENTS FOR CONCRETE.

The chief requirements for concrete are strength, watertightness, and appearance. Especially in the strength and in the watertightness does the quality of the aggregate play a most important part.

The strength is affected by:

(a) Strength of the particles. If, for example, a soft material like chalk is used for the aggregate, it is evident that the concrete can have but little ultimate strength.

(b) Adhesion of cement to the particles. If the grains are not strongly cemented, the mass cannot attain high strength. This is an important point in connection with aggregates, because a film over the surface of the grains will reduce the adhesion and, consequently, the strength of the concrete.

(c) Density. It is evident that a porous material has less strength than a dense one. It is not always remembered, however, how far this is actually true in concrete and the tremendous influence that the selection and proportioning of the aggregate have upon this quality.

Watertightness requires:

(a) A non-porous aggregate. Cinder concrete, for example, cannot be expected to be as watertight as stone or gravel concrete.

(b) Adhesion of cement to aggregate. The stronger the adhesion, the greater the resistance to penetration of water.

(c) Smallness of voids. In strength, the percentage of voids exercises the greater influence, while in watertightness the size of the voids has the greater effect. Very minute voids, even if occupying a considerable percentage of space in the mass, do not permit the flow of water.

The appearance of concrete is affected by the aggregate by:

(a) Stone pockets which may be produced through improper selection or proportioning of the aggregate.

(b) Texture. A fine or a graded fine aggregate produces a finer texture than a very coarse sand.

COMMERCIAL TESTS OF FINE AGGREGATE.
(Laboratory of Sanford E. Thompson)

		Tensile Strength of 1:3 Mortar and Ratio of Strength to 1:3 Standard-Sand Mortar						Mechanical Analysis % passing sieves No.						
		3 Days		7 Days		28 Days								
		Lb. per Sq. In.	Ratio %	Lb. per Sq. In.	Ratio %	Lb. per Sq. In.	Ratio %	¼″	12	40	100	200		
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)		
(1)	Standard Sand	175	100	230	100	315	100	Passed No. 20 and retained No. 30					From Ottawa, Ill.	
(2)	Near Portland, Maine	00	00	00	00	85	27	100	83	22	5	3.1	2½ ft. deep	Sand at different depths from sand pit. Discarded and Sand (9) substituted.
(3)	" " "	46	27	113	49	121	38	100	80	13	0.96	0.45	4 ft. "	
(4)	" " "	34	20	85	37	95	30	100	89	24	1.3	0.46	5 ft. "	
(5)	" " "	38	22	90	39	105	35	100	78	18	1.0	0.5	5½ ft. "	
(6)	" " "	54	31	120	52	123	39	100	87	24	1.2	0.26	7½ ft. "	
(7)	" " "	62	36	113	49	146	46	100	79	23	2.4	0.83	8 ft. "	
(8)	" " "	----	72	163	71	220	71	100	88	23	0.65	0.26	9½ ft. "	
(9)	" " "	190	110	241	105	385	122						Sand from neighboring pit.	
(10)	Bangor, Me.	100	57	168	73	237	75	100	83	27	7.1	3.9	3 ft. deep	
(11)	" "	139	81	244	106	322	102	100	65	17	4.3	2.5	6 ft. "	
(12)	" "	155	90	271	118	383	121	100	52	8	2.0	1.0	11 ft. "	
(13)	Milford, N. H.	......	....	32	14	63	20						Building failed. Impure sand	
(14)	" "	......	....	196	85	300	95						New sand used in rebuilding	
(15)	Wilton, N. H.	00	00	25	11	69	22	100	99	92	9.5	2.3	Discarded	
(16)	" "	141	82	221	96	284	90	100	84	18	1.2	0.45	Accepted	
(17)	Boston, Mass.	121	70	228	99	332	105	100	81	23	4.9	2.2		

(18)	Salisbury Beach, Mass.	157	91	219	95	310	98	100	93	27	0.6	0.0	
(19)	Bridgewater, Mass.	100	58	161	70	329	104	100	93	58	15	4.9	Dirty. Screened from gravel
(20)	" "	38	22	124	54	322	80	100	78	26	5.6	2.9	Good sand pit
(21)	Sharon, Mass.	110	64	221	96	313	99	100	86	46	6.2	1.6	Too fine
(22)	Webster, Mass.	188	109	260	113	395	125	100	62	27	7.1	2.8	Commercial bank
(23)	Maynard, Mass.			230	100	405	128						
(24)	Boston, Mass.	68	39	113	49	186	59	100	60	16	1.8	0.5	Building site rejected
(25)	" "	128	74	242	105	362	114	100	96	55	1.56	0.62	Same building site accepted
(26)	" "	187	108	295	128	477	151	100	88	26	1.6	0.6	Commercial bank
(27)	" "	103	60	212	92	305	97	100	96	32	7.8	2.9	" "
(28)	" "	45	26	92	40	143	45	100	92	43	2.6	0.9	Dredged. Impure. Rejected
(29)	" "			108	47	215	68						Cornice failed
(30)	" "	26	15.5	145	63	199	63						Wall failed. Impure sand
(31)	Pittsfield, Mass.			205	89	332	105						Rejected bank
(32)	" "			152	66	221	70						Accepted bank
(33)	Forest Hills, Mass.			53	23	142	45	100	68	9	1.9	1.2	Failed to harden. Impure sand
(34)	" " "	154	89	225	98	330	104						Accepted
(35)	Saylesville, R. I.			37	16	143	45						Rejected. Impure sand
(36)	" "			191	83	247	78						Same sand washed
(37)	" "			168	73	280	89						Same sand. Different brand of cement
(38)	Pawtucket, R. I.			225	98	437	138	100	78	27	4.2	1.5	
(39)	Hartford, Conn.	85	49	140	61	161	51	100	82	45	10.4	5.8	Too fine. Rejected
(40)	" "	155	90	250	109	316	100						Accepted
(41)	Falls Village, Conn.	190	110	265	115	370	117	100	46	32	14.6	7.4	

COMMERCIAL TESTS OF FINE AGGREGATE—Continued.
(Laboratory of Sanford E. Thompson)

		Tensile Strength of 1:3 Mortar and Ratio of Strength to 1:3 Standard-Sand Mortar						Mechanical Analysis % passing sieves No.					
		3 Days		7 Days		28 Days							
		Lb. per Sq. In.	Ratio %	Lb. per Sq. In.	Ratio %	Lb. per Sq. In.	Ratio %	¼″	12	40	100	200	
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	
(42)	New York C. (Cowe Bay sand)	261	151	258	112	311	98	100	86	31	2.07	1.58	
(43)	Watervliet, N. Y.	184	107	262	114	363	115	100	84	46	9.6	3.2	
(44)	Garrison, N. Y.	268	155	290	126	414	131	100	55	16	5.8	3.6	
(45)	Kensico, N. Y.	133	77	205	89	276	87	100	71	45	26.9	14.3	Screenings
(46)	E. Orange, N. J.			126	55	291	92	100	99	94	15.0	4.7	Screenings
(47)	Frankfort, Penn.	00		25	11	82	26	100	99	66	7.0	1.08	Used in Edison work
(48)	" "	91	52	188	82	315	100						Building failed. Impure sand
(49)	Philadelphia River Gravel	209	122	294	128	347	110	100	70	22	1.8	1.0	Sand from gravel
(50)	" " "	175	101	255	111	327	103	100	75	31	2.9	1.10	
(51)	Philadelphia Jersey Gravel	72	42	117	51	165	52	100	92	55	8.1	0.9	
(52)	Bridgeport, Penn.	128	74	198	86	313	99	100	86	45	6.1	1.5	Slow hardening
(53)	Philadelphia Rapid Transit Com.			172	75	291	92	100	75	38	5.7	3.6	
(54)	" " " "			244	106	320	101	100	74	36	3.7	0.4	
(55)	Waupaca, Wis.	230	134	322	140	443	140	100	87	25	0.8	0.16	Washed
(56)	Keokuk, Iowa	175	101	253	110	320	101	100	82	14	1.04	0.14	Exceptionally clean and pure
(57)	Tennessee River	62	36	103	45	184	58	100	97	63	2.3	0.8	River sand. Fine

(58)	Knoxville, Tenn.	292	169	386	168	592	187	100	43	16	5.2	2.1	Screenings commercial
(59)	Alfia River, Fla.	110	64	156	68	193	61	100	99	89	8.5	0.7	
(60)	Medina, Tex.			388	169	557	177						Limestone screenings
(61)	Calaveras, Cal.			158	69	259	113						
(62)	St. Johns, N. B.	101	58	180	78	224	71						Impure but very coarse
(63)	Nova Scotia	48	28	92	40	177	56	100	100	97	17.5	0.5	Very fine
(64)	" "	196	114	250	109	308	97	100	31.5	2.7	0.9	0.4	Coarse
(65)	Ontario, Can.	391	226	483	215	592	187	100	88	10	1.0	0.6	Limestone composition
(66)	Cedar Rapids, Can.	59	34	90	39	168	53	100	97	66	6.6	1.8	Clean but fine
(67)	" " "	314	182	308	134	465	147	100	51	9	3.7	1.8	Limestone sand
(68)	Calgary, Can.			163	71	318	101	100	92	47	11.4	5.0	
(69)	England	137	79	235	102	255	82	100	83	30	0.8	0.16	Concrete sand

EFFECT OF THE AGGREGATE UPON QUALITY AND STRENGTH OF CONCRETE AND MORTAR.

Tests of aggregates may be considered in two general classes:

1. Tests of quality
2. Tests for acceptance

These divisions frequently merge into one another, because the special characteristics of an aggregate may render it unfit for certain work, although it satisfactorily passes a standard test for acceptance. In other words, each material must be considered not simply alone, but in its relation to the conditions of use and to the other materials of the mortar or concrete.

The variation in strength of both mortar and concrete due to the quality of the aggregates, especially of the fine aggregate, is insufficiently recognized. To illustrate the variation in strength of mortars made with different sands, the following table is drawn up from some of the recent commercial tests made in the author's laboratory. All of the aggregates shown have been used in concrete or proposed for use. Following many of the tests, there is given the probable reason for the variation in strength from the normal. It will be noticed that both the actual tensile strength in pounds per square inch and the ratio of strength of the mortar to the strength of standard sand mortar are given. In a number of instances there is given the test of a poor sand followed by that of a good sand obtained from the same locality. The mechanical analysis is added to illustrate, in many cases, the effect of the sizes of the grains upon the strength. The total percentages passing each sieve are given in each analysis.

As a further illustration of the variation in quality of sands, the report of the Committee on Specifications and Methods of Tests for Concrete Materials of the American Concrete Institute, 1913, states that in tests in the laboratory of one of the members of the committee, "Out of 37 samples of sand from different parts of the country made into 1:3 mortar, 12 were equal to or greater in strength than similar mortar of standard Ottawa sand; 5 were between 90 and 100 percent of standard Ottawa sand; 6 were between 80 and 90 percent; 4 were between 70 and 80 percent; 6 were between 60 and 70 percent; 1 between 50 and 60 percent; 2 between 40 and 50 per cent; and 1 between

30 and 40 per cent. Such results as these indicate the absolute necessity for always testing sand that is used in concrete''.

TESTS OF QUALITY.

The characteristics of aggregates to be considered as affecting the quality of the concrete or mortar are:

(1) Mineralogical composition
(2) Specific gravity
(3) Weight
(4) Voids
(5) Shape of grains
(6) Coarseness
(7) Gradation in size of grains, i. e., mechanical or granulometric analysis
(8) Density of mortar or concrete
(9) Impurities.

(1) Mineralogical Composition. The mineralogical composition of the aggregate affects the shape and size of grains, the specific gravity, and in some cases its chemical action with the cement.

It is evident, as has been stated already, that a soft aggregate will produce weak concrete. A shale or a very soft limestone or sandstone is a very poor aggregate. The Joint Committee on Reinforced Concrete gives in its report for 1913* the following strengths for concrete made with different kinds of aggregate, which are recommended as maximum ultimate values.

Strengths of Different Mixtures of Concrete.
(In pounds per square inch)

Aggregate	1:1:2	1:1½:3	1:2:4	1:2½:5	1:3:6
Granite, trap rock	3300	2800	2200	1800	1400
Gravel, hard limestone and hard sandstone	3000	2500	2000	1600	1300
Soft limestone and sandstone	2200	1800	1500	1200	1000
Cinders	800	700	600	500	400

A quartz sand is apt to be the best for concrete, not so much because of its mineralogical nature as because a quartz material is apt to be well graded, with comparatively little dust or silt.

* Transactions American Society Civil Engineers, Vol. LXXVII, p. 385.

In certain localities good sand contains considerable trap. Sands of limestone composition are apt to produce concretes and mortars of higher strength than normal, probably because of chemical action with the cement.

It will be noted in the preceding table that the limestone sand and screenings run very much higher in strength than a standard-sand mortar.

In tests made by the author for the selection of materials to use in a large dam in Texas, a 1:4:7 mix, by using graded aggregate of limestone with enough dust in the fine aggregate, was found to give a strength equal to 1:3:6 proportions using first-class sand and gravel.

Mica in the form of laminated grains reduces the strength of mortar.

(2) Specific Gravity. Determination of the specific gravity of aggregates is necessary for the calculation of the absolute volume and density of the mortar or concrete. The specific gravity of rock of the same class varies greatly. For example, trap may vary from 2.78 to 3.03, limestone from 2.50 to 2.80, granite from 2.60 to 2.80, and sandstone from 2.10 to 2.75.

The specific gravity of natural sand, however, is quite uniform and may be assumed for ordinary computations of weight and voids as 2.65. For density experiments of mortar and concrete more exact determinations should be made in each case.

For determining the specific gravity of sand, a specific gravity apparatus such as the Jackson flask is convenient. For coarse aggregate, pieces may be suspended in water on laboratory scales. Specific gravity of coarse aggregates is of some value as indicating the hardness and therefore the strength of the particles. In general, a high specific gravity indicates a hard stone.*

(3) Weight. The weight per cubic foot of any aggregate is dependent upon the specific gravity of the particles and upon the voids. In sand the difference in the percentage of moisture and the methods of handling so affect the weight that it is of little value except for comparative tests. In general, the heavier the sand or stone, the better it is, because it indicates greater density.

* Approximate average values of various aggregates are given in Taylor and Thompson's "Concrete, Plain and Reinforced", 2nd Ed., p. 163.

The weight of bank sand varies greatly with the moisture. Fine sand retains moisture, which tends to separate the grains and increase the voids, and may run as low as 80 pounds per cubic foot (including moisture). As sand becomes coarser, it contains less moisture and may weigh up to 110 pounds per cubic foot. An average weight for bank sand loose is about 95 pounds per cubic foot.

(4) Voids. Considered as a characteristic by itself, the percentage of voids in an aggregate is of no value. Formerly, aggregates for concrete were proportioned by determining the voids in the material, then computing the quantity of next finer material to fill these voids, and so on down to the cement. As a matter of fact, however, in practice the grains of one material are seldom small enough to exactly fit into the voids of the next larger size, so that they force apart the coarser particles to a greater or less degree, and the amount of such expansion depends entirely on the relative sizes of the grains. This method of proportioning, therefore, is absolutely incorrect.

Furthermore, the percentage of voids in dry sand is of scarcely any practical value because the condition of moisture so greatly affects the voids. Tests by Wm. B. Fuller* show a variation in the same sand under different conditions of moisture of from 33% voids to 44% voids. A minimum percentage of voids occurs when the sand is either completely saturated or else is absolutely dry. The maximum percentage of voids, as well as the percentage of moisture ordinarily retained in the bank, varies with the sizes and relative sizes of the grains of the sand.

A mass of equal spheres systematically piled in the most compact manner contains 26% voids. In practice the voids in fine aggregate may vary from 30% to 50%, including both moisture and air.

In minimizing the effect of voids in an aggregate, it must not for one moment be understood that the voids in the final mixture of aggregates and cement do not exert an immense influence upon the strength and watertightness of the concrete. The difficulty lies in obtaining some means for testing the voids of the aggregates individually, or even in combination, so as to indi-

* See "Concrete, Plain and Reinforced", by Taylor and Thompson, 2nd Ed., p. 177.

cate their effect upon the final density of the concrete or mortar. The quality of density is referred to below.

(5) Shape of Grains. The shape of grains is of importance simply as it affects the density of the mixture. A fine or coarse aggregate composed of laminated, i. e., flat grains, will not pack so closely in the mass, so that there will be more voids and less density. Rounded grains pack closer than angular grains, and for this reason the strength of a mortar or concrete composed of aggregates with rounded grains is frequently higher than that produced with sharp-grained aggregate.

(6) Coarseness. A coarse sand produces a stronger mortar than a fine sand. In a concrete of rich proportions the density and strength of the mass are increased as the maximum size of the coarse aggregate increases, provided the mixture is homogeneous and the gradation of grains is similar. This is illustrated in the following table:*

Aggregate	Average Density	Ratio of Densities	Modulus of Rupture Ratios of Strength at 90 days	Compressive Strength at 140 days
2¼″ stone........	0.847	1.00	1.00	1.00
1″ "	0.814	0.96	0.91	0.83
½″ "	0.788	0.93	0.75	0.72

The effect of size of grain upon mortar made with aggregate of different fineness is illustrated by tests of L. C. Sabin.†

Tests with Limestone Screenings of Varying Fineness.

Approximate mean size of grain	Designation of size by sieve numbers	Ratio of tensile strength Av. of 6 mo., 12 mo. and 4 yr. tests
0.057	10–20	1.00
0.038	20–30	0.93
0.020	30–40	0.84
0.015	40–50	0.72
0.012	40–80	0.66
0.008	pass 50	0.59

The effect of different coarseness is also illustrated by the tables, already given, which show the mechanical analyses of a

* "The Laws of Proportioning Concrete", by Fuller and Thompson, Transactions American Society of Civil Engineers, Vol. LIX, 1907.

† Sabin's "Cement and Concrete", p. 160.

number of different sands and the tensile strength of 1:3 mortars made with them.

The effect of moisture in sand upon the percentage of voids has been referred to. In mixing mortar and concrete a similar principle is involved, as it is found that a fine sand requires more water than a coarse sand. The resulting mortar with fine sand therefore occupies more volume and, consequently, is less dense than the mortar from coarse sand. This is the reason

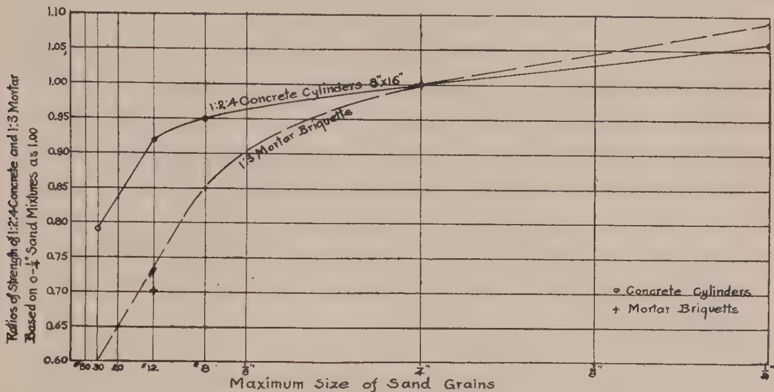


Fig. 2. Relative Strengths of Mortars and Concretes Due to Difference in Coarseness of Sand. Based on Strength of Specimens Mixed With 0-¼-in. Sand as 1.00.

Average Age { **8″ x 16″ Concrete Cylinders—2 Months.**
Mortar Briquettes—28 Days.

why a coarse sand produces a stronger mortar than fine. Mr. Féret says:* "Tests indicate that every size of sand requires a definite percentage of water for gaging it to plastic consistency" and that "sands of different granulometric composition require a percentage of water varying from 3% to 23%".

The coarseness of the fine aggregate greatly influences the strength of concrete as well as mortar, although to a less degree. Fig. 2, from the report of the Committee on Specifications and Methods of Tests for Concrete Materials of the American Concrete Institute, 1915, gives curves of ratios of strength of mortar and of concrete made from the same aggregate.

* Annales des Ponts et Chaussées, 1892, II. See also Taylor and Thompson's "Concrete, Plain and Reinforced", 2nd Ed., p. 179.

Basing the ratios of strength on the strengths of mortar and concrete made with sand running up to $\frac{1}{4}''$ in size, the relative strengths using sand of different maximum size are illustrated.

(7) Gradation in Size of Grains. The theory of proportioning concrete and mortar requires a gradation of grains of the aggregate so that the particles will fit into each other in such a way as to produce the densest mixture. It has been shown that with the same percentage of cement and different arrangements of aggregate the strongest concrete is usually that in which the aggregate is proportioned so as to give concrete of the greatest density, that is, with the smallest percentage of voids. Therefore, the aim in proportioning is to attain this result.

The space allotted to this paper is too limited to treat in detail of the laws of proportioning aggregates. Researches have been extended far enough to show that with a given percentage of cement the strength of concrete or mortar bears a definite relation to the density. Mr. Féret has evolved a formula* for strength of mortar based on the absolute volumes of solid materials composing it. The author of this paper has applied similar principles to concrete proportioning.†

Extended series of tests by Messrs. Fuller and Thompson at Jerome Park Reservoir‡ indicate that there is no real line of demarcation between coarse and fine aggregate, but that the effect upon the mixture of the concrete depends upon the gradation of grains of the entire mixture. Particles with a diameter less than 1/10 that of the maximum size act really as fine aggregates. The true meaning of the term "fine aggregate", therefore, is dependent upon the maximum size of the coarse aggregate.

In conformity to this law, the author finds that sand for concrete requires more fine material than mortar sand. Tests indicate that the best percentages passing a No. 40 sieve may range from about 18% for a 1:2:4 concrete up to 27% for a 1:4:8

* Bulletin de la Société d'Encouragement pour l'Industrie Nationale, Vol. II, p. 1593, 1897. See also Taylor and Thompson's "Concrete, Plain and Reinforced", 2nd Ed., p. 140.

† Taylor and Thompson's "Concrete, Plain and Reinforced", 2nd. Ed., p. 355.

‡ Transactions American Society of Civil Engineers, Vol. LIX, 1907.

concrete. For water-tight concrete, even a larger percentage of fine grains appears to be beneficial.

Tests for gradation in size of grain are made by methods of mechanical analysis. This involves passing the aggregate successively through a series of sieves of different mesh and recording the total percentage passing each sieve. Illustrations of mechanical analyses are given in the foregoing table of tests of sands made in the author's laboratory.

(8) Density of Concrete and Mortar. The effect of density upon mortar was first brought out by Mr. Féret, and its influence upon the concrete, by Mr. William B. Fuller. Various tests have repeatedly proved the fact that, other things being equal, the densest concrete and mortar have the highest strength. For this reason it is possible to select aggregates for mortars and concrete by making density tests of the materials mixed with cement and water in the proportions to be used on the job.*

(9) Impurities. Illustrations of the effect of impurities in fine aggregate are shown in the preceding table of tests. Several of these samples, notably Nos. (13), (30), (33) and (47), illustrate cases of actual concrete failures due to the impure sand.†

A sand which is too fine or poorly graded may produce a mortar of low strength. A sand which is impure may produce a mortar or concrete which will never attain strength and, in fact, which may never properly harden. Impurities which give the most trouble are of organic origin, usually of a vegetable nature. Case after case has been brought to the author where sands, good in appearance and apparently satisfactory, have been used in concrete which failed to harden because of a very small quantity of impurities of this nature. Frequently these impurities are in the form of a coating on the grains of sand.

TESTS FOR ACCEPTANCE.

The above discussion of the variation in strength of mortars made with different sands and the table of actual tests

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* See "Laws of Proportioning Concrete", by Fuller and Thompson, Transactions American Society of Civil Engineers, Vol. LIX, 1907.

† The experience at Milford, N. H., is described in a discussion on "Impurities in Sand for Concrete", Transactions American Society of Civil Engineers, Vol. LXV, 1909.

show very positively that sand or other fine aggregate should always be tested. The most satisfactory test up to the present time is the determination of the strength of mortar made up with the cement and fine aggregate to be used on the work in comparison with the strength of identical specimens made in the same proportions with standard sand. The results are given in terms of a ratio or percentage.

The requirements of the Joint Committee on Concrete and Reinforced Concrete are as follows:

> "Fine aggregates should be of such quality that mortar composed of one part Portland cement and three parts fine aggregate by weight when made into briquets will show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand. If the aggregate be of poorer quality the proportion of cement should be increased in the mortar to secure the desired strength.
>
> "If the strength developed by the aggregate in the 1:3 mortar is less than 70 per cent of the strength of the Ottawa-sand mortar, the material should be rejected. To avoid the removal of any coating on the grains, which may affect the strength, bank sands should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture may be determined upon a separate sample for correcting weight. From 10 to 40 per cent more water may be required in mixing bank or artificial sands than for standard Ottawa sand to produce the same consistency".

For requiring special characteristics, the fine aggregate must be accepted not merely on passing the tests for strength, but for tests such as those for permeability of the mortar and concrete; the effect of different brands of cement; the effect of frost action; and the effect of fire. Characteristics of yield, density, chemical composition, mechanical analysis, and amount of organic matter, are frequently required.

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DISCUSSION

Mr. J. A. Kitts,* Assoc. M. Am. Soc. C. E., calls attention to the following statement made by Mr. Thompson in speaking of voids and proportioning with regard to the voids in the materials, ''this method of proportioning, therefore, is absolutely incorrect''. Mr. Kitts.

Mr. Kitts has made some tests on the Panama Canal with particular regard to the voids and can state that, insofar as Mr. Thompson assumes that any method of proportioning with regard to the voids is incorrect, he is positively in error.

* Berkeley, Cal.

Mr. Kitts. The Panama Canal tests were made by the weight-volumetric method of proportioning,* which consists of finding accurately, by several determinations, the specific gravity and percentage of voids of the materials, from which determinations the apparent specific gravity is computed; and, by use of the apparent specific gravity, weights of materials may be computed for any desired volumes or proportions. The equations involved are as follows:

$$\text{Volume} = \text{weight} \div \text{specific gravity} \quad \ldots \quad (1)$$

$$\text{Apparent volume} = \text{weight} \div \text{apparent specific gravity} \quad \ldots \quad (2)$$

$$\text{App. sp. gr.} = (\text{unit app. vol.} - \text{voids}) \text{ specific gravity} \quad \ldots \quad (3)$$

$$\text{App. vol.} = \text{weight} \div (\text{unit app. vol.} - \text{voids}) \text{ sp. gr.} \quad \ldots \quad (4)$$

If it is desired to use a proportion of cement paste equal to the proportion of voids in the particular sand, the weight proportions corresponding to the assumed volumetric proportions are determined by the following equation:

$$\frac{\text{Weight of cement}}{\text{Unit weight of sand}} = \frac{\text{App. sp. gr. cem.} \times \text{prop. voids in sand}}{\text{App. sp. gr. sand}} \quad (5)$$

If several sands are being studied, the ratio of cement paste to voids in the sand may be varied, for example, from 0.85, 0.90, 0.95 to 3.0 etc., by putting this figure in the numerator of equation (5). The sands with 85% of the voids filled with cement paste are comparable; the sands with 90% of the voids filled with cement paste are comparable, and so on. Tests of cement-sand mortars, with various sands, in the usual 1:3 weight proportions are not comparable, as the ratios of cement paste to voids in the sands may vary from 50% to 150%; such tests are of doubtful practical or scientific value and have contributed little to the knowledge of the laws of mixtures.

Proportioning for concrete may be done in a similar manner. A volume of rock equal to that of the test cylinder may be used for each test specimen.

$$\text{Wt. of rock to fill cyl.} = \text{App. sp. gr. rock} \times \text{vol. of cyl.} \quad \ldots \quad (6)$$

$$\text{Vol. of voids} = \text{Vol. cyl.} \times \text{prop. voids in rock} \quad \ldots \quad (7)$$

$$\text{Weight of sand} = \frac{\text{Vol. of mortar} \times \text{app. sp. gr. sand}}{\text{Yield of mortar}} \quad \ldots \quad (8)$$

$$\text{Weight of cement} = \frac{\text{Vol. mortar} \times \text{prop. vds. in sd.} \times \text{app. sp. gr. cem.}}{\text{Yield of mortar}} \quad (9)$$

The "volume of mortar", equations (8) and (9), may be taken as equal to the volume of voids in the measure of rock, or as a fraction or multiple of same, if it is desired to use progressive proportions of mortar with the rock. If it is desired to use a mortar in which the volume of

* "Weight-Volumetric Proportioning of Concrete Aggregates in Testing", J. A. Kitts, Proceedings, Am. Soc. Test. Mats., Vol. XV, Part II, pp. 153-170 (1915)

cement paste is equal, for example, to 115% of the voids in the sand, 1.15 should be used as a multiple in the numerator of equation (9). The "yield of mortar" is determined by experiment and should be expressed as a multiple of the apparent volume of sand used, this volume of sand being taken as unity. Mr. Kitts.

These Panama Canal tests showed that an amount of cement paste equal to the voids in the sand gave a mortar of minimum cost per unit of strength and of maximum density, and that with an amount of mortar equal to the voids in the rock, a concrete of minimum cost per unit of strength and of maximum density was obtained.

Prof. Withey's tests of mortars referred to by Mr. Thompson were made in 1:2, 1:3, 1:4 and 1:5 weight proportions. A chart was made by Prof. Withey showing the relative cost per unit of strength of the four mixtures of each particular sand. Mr. Kitts computed the ratios of weights of cement to sand that would give a volume of cement paste equal to the voids in the sand and in 9 out of 14 cases his proportions agreed with Prof. Withey's results.

The evidence shows that proportioning with regard to the voids is correct and it is reasonable to suppose that a method based on physical science, as we know it, is more apt to be correct than any arbitrary method of proportioning, such as 1:2:4, 1:3:6, etc., or any empirical and mechanical method of proportioning, such as proportioning by screen analysis.

Mr. Muckleston. **Mr. H. B. Muckleston,*** M. Am. Soc. C. E., stated that he knew of one particular case where a certain concrete was proportioned so that there were only 16½% of voids in the mixture, and 1½ bbls. of cement to 1 yd. of gravel were used; but the resulting concrete was found to be worthless and would crumble in the fingers.

Upon making a chemical analysis, it was found that strong tannic acid crystals prevented the setting of the concrete. This shows that a chemical analysis is sometimes necessary to determine the properties of an aggregate.

Mr. Graf. **Mr. S. H. Graf,†** Jun. Am. Soc. M. E., said that all cities test cement, but hardly any make tests of sand.

Ninety-five percent of the failures of concrete are due either to poor workmanship or to the weakness of the aggregate. Several weeks, or even months, may be required for testing an aggregate and it should not be left to the last minute. For example, in Tillamook County, Oregon, a concrete highway was contemplated. The sand for this job was found, after five or six tests, to be deficient. The best sand developed only 35% of the strength developed by standard Ottawa sand. All bids were rejected, because all contractors had figured on using the local sand.

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Mr. Graf. Comparative tests of sand samples are not always fair tests, if the proportions are taken by weight. For example, tests of 30 different samples were made for the State of Oregon by making one to three mortars, proportioned by weight; but the weights of these different samples varied from 75 lbs. per cubic foot to 150 lbs. per cubic foot; consequently, the comparisons were unfair, especially as the heaviest sand proved to make the weakest mortar. Tests made from specimens proportioned by volume showed better the true nature of the sand samples.

Mr. Thompson. **Mr. Sanford E. Thompson,** M. Am. Soc. C. E., in closing, said that the discussion of the characteristics of voids by the author of the paper was intended to call particular attention to the need of taking into consideration the actual and the relative sizes of grain and the effect of moisture in practice, and to show that these so affect the results of any sets of tests that, as stated, "considered as a characteristic by itself, the percentage of voids in an aggregate is of no value". Undoubtedly, as Mr. Kitts indicates, tests of voids for comparative examinations of aggregates under certain conditions are of considerable value, and I would refer the reader to our book, "Concrete, Plain and Reinforced" for a more complete treatment of the entire subject.

Paper No. 102

PROBABLE AND PRESUMPTIVE LIFE OF CONCRETE STRUCTURES MADE FROM MODERN CEMENTS.

By

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Some concrete casually made by empirical means has in some cases stood the test of 2000 years or more. It is fruitless to enquire as to the proportion which went to pieces.

Today we know with some approach to precision how to make concrete, and according to the exercise of our knowledge, our structures will last as long as there is any need for their existence, for it is the fate of all modern structures to be torn down and replaced within periods reckoned not by hundreds of years but by units of ten.

Since the time when Portland cement became accepted as one of the principal materials of construction, doubts were felt as to its permanence. These doubts in the beginning were thoroughly justified. A material prepared in the crudest way, with wholly inadequate knowledge and that picturesque method of control, known as rule of thumb, and from its nature and constitution, resembling a slow but highly effective explosive, would be false to its genesis and properties if it failed to provide a plentiful crop of disasters—and this it did.

Nowadays the situation is very different, not because modern cement is stable any more than is any other structural material, but because the properties, uses and limitations of cement are fairly well understood, and a reasonable amount of care is exercised in its employment. It would be wrong to assume that our present knowledge and skill are sufficient to relieve us of anxiety and occasional calamity in erecting structures intended to be permanent, but we are no longer in the position of speculators in a rather gambling stock, expectant of a fall.

In order to discuss the question of the probable life of concrete structures made with modern cement, it is necessary to consider the chief causes which determine that life.

I have drawn up a list of such causes, which, though not necessarily exhaustive, may serve as a guide to an orderly sequence in the present paper. For convenience in this list, ordinary concrete is catalogued separately from reinforced concrete, although, of course, in the majority of cases what affects one affects the other; the probable life of reinforced concrete is that of the concrete and the reinforcement severally and jointly, its characteristics as a material *sui generis* influencing in one direction or another its capability of continuing existence.

There is, of course, no such thing as a permanent structure of whatever material it may be made, and the present enquiry is limited to the consideration of how far concrete may be regarded as subpermanent. I do not think that anybody here will deny its right to rank with the most resistant structural materials known, and there are many—and I include myself—who believe that it stands very high in that rank if applied with knowledge to its proper purposes. The list of possible causes of destruction, mentioned above, is as follows:

A. Ordinary concrete as distinguished from reinforced concrete:

- (1) bad cement,
- (2) bad aggregate,
- (3) bad proportions,
- (4) bad mixing,
- (5) bad workmanship,
- (6) bad design,
- (7) external violence,
- (8) fair wear and tear,
- (9) action of saline solutions,
- (10) action of acids,
- (11) electrolysis.

B. Reinforced concrete.

All the foregoing causes of destruction are operative towards reinforced concrete as well as plain concrete. In addition there are:

(1) corrosion of reinforcement direct or by electrolysis,

(2) cracking due to monolithic character or possibly to stresses between the concrete and the reinforcement.

The term "modern cement," which forms part of the title of this paper, should rule out bad cement, but unfortunately it does not. The best modern cement made of suitable raw materials, intimately mixed, thoroughly burnt and finely ground, is as dependable a material as can be prepared until the time comes when all cement is made by fusing the constituents in a sort of super-blast-furnace, a method tried some years ago in the United States, and one which I have long regarded as the rational advance on the present rotatory process. But these conditions of excellence are not always fulfilled. Chiefly because of the endeavour to obtain large outputs of cement per unit of plant, the control of proportions is sometimes inaccurate, the burning not uniform and the grinding not only coarser than is desirable but "gritty." Such cement fails in respect of the first quality absolutely essential to the stability of any structure of which it forms part—it is not sound. Quite useless is it to say that such unsound cement has been used and the structures made with it are standing; the point of interest is how many have fallen down. Further there is the pregnant question whether a buyer will not insist on a material which is certain to be free from vice, or whether for the convenience of the seller he will trust to luck. Generally, the man who pays can and will get what he wants. It may be confidently said that given careful manufacture, rigid inspection and thorough testing to a searching specification, modern cement can be obtained free from all inherent vice, and that structures of which it forms part will not be brought to a premature end by internal treachery.

Bad aggregate is a fruitful source of trouble, and, simple as it is in a specification to say that the aggregate shall be "suitable, clean, sharp, well washed", and so on, it is not always easy to get such an aggregate at a reasonable price. Local material must almost always be used, and it may be of the most diverse description. The one property, which is indispensable, is that it must be chemically stable under the con-

ditions in which it is to be used. It does not follow absolutely that the aggregate shall be stable *per se,* though it is much better that it should be; there are materials which oxidise, or which weather, that may on occasion act as a serviceable aggregate, but only urgent necessity will sanction their use. Thus, in general, rocks containing pyrites should be avoided, but it would be pedantic to reject a granite or a hard limestone on the sole ground that specks of pyrites are present. Not merely the amount and size of the enclosed pyrites should be considered; naturally a rock containing marcasite is *ipso facto* suspect. In such cases, petrological methods of examination should be used. Similarly, slags, such as copper slag containing much ferrous silicate, may well be used if their silica content is high enough; generally, such slags lie in dumps, and have so lain for years, and their behaviour during exposure to weather is a great guide The same remark applies to blast furnace slag. Analysis is very helpful if the results are carefully interpreted, but the behaviour of the material on the dump is even better. I have seen a considerable quantity of concrete destroyed by the use of a slag aggregate containing calcium sulphate derived from calcium sulphide in a fairly basic blast-furnace slag, and in this instance analysis would have detected the destructive agent, and saved much trouble and expense. Speaking generally, substances containing sulphates or sulphides, capable of oxidation under working conditions, are so dangerous that their use should not be tolerated, and the need of this restriction can be the better realized when it is remembered that 1% of SO_3, calculated on the aggregate, may mean 5% or more on the cement. Perhaps, of all the materials used as aggregate, the most dangerous is coke breeze. Mind, I do not say injurious, I say dangerous. The danger lies in the fact that some samples contain an abundance of sulphates, and, on account of the porous nature of the breeze, these are readily extracted, and do their deadly work on the cement. No sample of breeze should be used as an aggregate unless it has been analyzed and tested. Unless the strictest supervision of this kind is exercised, disaster is courted. Aggregate may be mechanically as well as chemically bad, but exactly how to define that badness is not easy. Such obvious defects as softness, cracks or excessive smoothness need no more than

mention, but how far a "dirty" aggregate carries its own condemnation, is a more difficult matter to decide. It may safely be said that clayey matter round the coarser lumps will prevent a proper bond, but the effect of a moderate amount of clayey matter in the sand is not necessarily harmful. Like most practical things, it is eminently a matter to be settled by trial, and test cubes of the proposed aggregate compared with similar cubes of some aggregate recognised as a standard, such as granite chips and clean sand, will decide the point. Four other causes of short life for a concrete structure, viz., bad proportions, bad mixing, bad workmanship and bad design, call for little comment except this, that evil as are all these for ordinary concrete, they are ten times worse for reinforced concrete, because, while ordinary concrete is generally used in considerable masses, a structure of reinforced concrete is a more delicate affair in which all four sources of mischief have a greater say. Particularly is this the case in respect of bad proportions and bad workmanship. All reinforced concrete should be as nearly impervious as can be contrived, as it is of the utmost importance to protect the reinforcement; and although it is true that iron is protected in an alkaline medium, yet reliance should not be placed on that alone; it is far sounder practice to make concrete of all kinds, and especially reinforced concrete, as nearly watertight as is practically possible.

The life of concrete structures may be shortened by causes which are external to itself. The violence of wind, wave and earthquake, the effect of the subsidence of the soil, in fact, all those things which in legal phraseology are called "the acts of God," because they are incomprehensible, will destroy any structure however well made. But, in practical affairs, one does not legislate for the infinite, and is content to make structures so good that ordinary natural violence will have little effect. The simplest and most important case is that of making harbours which must resist all these natural forces. Thanks to our harbour engineers, a fair degree of success has been attained, largely empirically. Putting aside for the moment the question of the quality of the cement, over which in former times they had little control, they understood in some degree that the concrete must be strong and dense, and, by proportioning the aggregate,

obtained a material which complied fairly with these requirements. But accurate measurement of voids and the knowledge that ordinary good concrete of about 1:6 may, and often does, contain 30% of voids, have not been so generally utilized as to prevent failures which are traceable to erosion and corrosion by the sea. It is not enough that a block of concrete should be strong; it must be as nearly as possible impervious and impenetrable. As I have said above, the need for these qualities in reinforced concrete is vastly more urgent; reinforced concrete has a vulnerable skeleton, and its exo-skeleton must be perfect. Fair wear and tear is only a mild case of external mechanical violence, and need not be further considered.

A particular form of external violence is the action of fire in any serious conflagration. It has been frequently stated that concrete structures are substantially fireproof, and, as far as inflammability is concerned, this is true, but it must be remembered that set cement is a substance containing combined water and carbonic acid, and that these are expelled at a comparatively moderate temperature. It might be naturally supposed that a structure exposed to fire would be seriously weakened by the decomposition of the essential cementitious constituents, and this surmise is of course correct. But, for all that, the amount of deterioration is less than one would think likely, and the appended table shows the results of a few experiments made on a cement mortar in the usual proportions of 3 to 1 by weight.

Test pieces were heated for 1 hour at the following temperatures:

Temperature Cent.	% Loss calculated on Cement.	
100	5.32	No appreciable effect.
200	14.12	" " "
300	16.68	" " "
400	16.56	" " "
500	17.96	" " "
600	21.92	Sound—weak at edges.
700	22.24	Sound—friable.
800	25.68	Sound—distinctly friable.
900	25.08	Sound—distinctly friable.
1000	24.36	Sound—very weak.
1100	25.40	Sound—very weak.

The fact that the last four figures do not show a progressive increase at the temperature rises is due to different test pieces being used, which, although of the same gauging, were necessarily not identical.

In no case, even at the highest temperature, were there any signs of disintegration or flying, and no mechanical loss occurred during the test.

Composition of test pieces, 3 standard sand to 1 Portland cement by weight. Age 3 months.

It will be seen that up to a temperature of 500° C. there is no appreciable alteration, and even beyond that the test pieces show considerable stability, a circumstance which is reassuring from the point of view of that most important question of fireproof construction. Before accepting such a conclusion unreservedly, however, it must be remembered that the tenderest members of reinforced concrete are the steel reinforcements, and that if the heat penetrates the envelope of concrete sufficiently to soften the steel, the destruction of the building will occur exactly as in the case of an ordinary steel frame building.

Shortening of the life of concrete by chemical action of external origin, which for the purpose of a list I have put under three headings, may be conveniently considered under one. A great number of investigators, including many honoured names, from the time of the pioneers Vicat and Michaelis to the time of those living, to whom it would be an impertinence to refer, have applied themselves to determine what is the probable or presumptive life of concrete, and, on account of the practical importance of the problem, have chiefly concerned themselves with the action of one saline solution. The destruction of concrete by seawater has always been, since the days when Portland cement first began to be used, a matter of much concern to engineers engaged in maritime works, and, even as lately as 30 years ago, much confusion of mind existed. Thus, because magnesia was found to be a predominant constituent of various incrustations and exudations on sea work, the erroneous conclusion was drawn that it was derived from the cement, and anxiety was felt concerning what could be considered the permissible limit for magnesia in cement. Of course, it is now common knowledge that the magnesia found has been formed

from the seawater by the action on it of the lime of the cement, and that the small quantities of magnesia normally present in Portland cement of good quality are without influence in these cases of injury. I do not propose to deal in detail with the destructive action on cement of magnesium salts and calcium sulphate in seawater, as the mechanism of the reactions in which they take part has been thoroughly studied by many able chemists, and their work is part of our general knowledge, but I propose to state shortly what my own analyses and experiments have led me to regard as the chief causes of the premature injury and shortened life of sea works made of concrete, and, in the first instance, I will deal with ordinary concrete.

It may be accepted that the heaviest and most important work is block work, and in this case the cement has ample time to harden before it is exposed to the sea. From consideration of expense, it is sometimes desired to use a comparatively poor mixture, but I cannot help thinking that the saving is dearly bought. In fact, the one indispensable condition for a long life for work exposed to the sea is the denseness and imperviousness of the concrete, and this is difficult to secure unless the cement is used liberally. It is impossible to fix a proportion, as that will depend on the aggregate. Every case must be judged for itself, the voids determined experimentally and enough cement used to fill them. Whenever any good form of puzzolanic material, such as trass and the like, is available it should certainly replace a part of the sand, for its use in forming a calcium silicate with the lime, normally set free during the setting of Portland cement, is undoubtedly of value, much conducing to the obtainment of that imperviousness which is a necessary condition for sound and lasting work. It should not be overlooked that any puzzolanic material can fulfil two functions. If coarsely ground, it acts partly as an aggregate like sand, and it is only when ground as finely as the cement itself that its full activity as a cementitious material comes into play. There is no objection to the use of coarse puzzolana if the supply is abundant and local, but, if it has to be brought from a distant place, it is evidently uneconomical to use part of it for a purpose equally well fulfilled by an inert material like sand. In some cases, it might

be desirable to grind the puzzolana and cement together to an equal fineness. This plan has been objected to by many engineers as being equivalent to an adulteration of the cement, but this view I hold to be mistaken if the mixture is sold under its own name, and the proportions of the two materials are stated. Many laudable attempts have been made to obtain imperviousness by the addition of the most various materials, such as barium salts, soap and fatty or mineral oils, but, though some of these are of value under special circumstances, they have not as yet shown themselves suitable for the heavy seawork now being spoken of; at present there is nothing better than ordinary concrete made with most carefully chosen and graded aggregate, with the addition of trass if local conditions allow, and an ample proportion of cement. Concrete made thus can only be attacked on the surface, and its destruction by percolation is well-nigh impossible. To state its probable length of life would be a rash attempt; in my view it should last indefinitely, in fact, until the harbour or other marine work had become obsolete.

When concrete has to be cast *in situ* opportunity for setting undisturbed is sometimes but poor, as compared with that of blockwork, but the same principles hold good, with the one addition that the setting time should be the minimum which will allow the material being got into position without disturbing or working it after setting has begun. Seawater is by far the most abundant saline solution, and contains those salts, magnesium salts and sulphates, which are most harmful to cement. What has been said of it applies to most other saline solutions which are likely to be harmful, and the precautions already mentioned apply in such cases. Of course, there are special instances of injury by such salts as sulphate of iron or the mixed metallic sulphates found in mine waters, but the nature of their attack is similar, and they are of too special a character to warrant more than mention in a paper dealing with the life of concrete structures in general. There is a common belief that salts in the act of crystallizing may expand and thus injure a structure of which they occupy the interstices. I have determined the amount of expansion of three typical, easily soluble salts and find the following results:

(1) Supersaturated solution of sodium sulphate.
Expansion on crystallizing..............1.45% by volume.

(2) Saturated solution of magnesium sulphate.
Contraction on crystallizing ....0.14% by volume.

(3) Supersaturated solution of sodium thiosulphate.
Contraction on crystallizing ....0.37% by volume.

I do not think that much importance need be attached to the view that concrete is injured materially by the crystallization of salts in its crevices, for the crystals—even when they do connote an increase of volume—are mechanically weak, and can exercise but little disruptive effect. It is the chemical action of saline solutions which is to be feared and guarded against.

Destruction of concrete by acids, and by this term acid salts are included, stands on a different footing. Obviously, strong acids turned to waste from a chemical works will destroy so calcareous a material as cement, and if the acid is sulphuric acid, destruction will proceed after the acid has been neutralized. But there are less obvious, though very real, causes of destruction. Many putrescent matters, such as sewage, will give off gases containing sulphur, and these, under suitable conditions, will oxidise and produce sulphurous acid, and, ultimately, sulphuric acid; or, alternately, will form sulphides, such as calcium sulphide or ferrous sulphide, which in due course, oxidise to the corresponding sulphates and injure or destroy any cement with which they may come in contact. It has been observed that with sewage of this kind flowing through concrete pipes the invert may be unaffected, while the arch is seriously attacked. The explanation generally accepted is that hydrogen sulphide, or some gaseous organic sulphide, is generated from the liquid, and coming in contact with the upper part of the pipe forms sulphides, which are oxidised to sulphates by the air above the level of the liquid. As the source of the sulphides, and therefore of the sulphates, is continuous, attack by the latter proceeds, with the result that the part of the pipe which is not immersed may suffer severe corrosion. To predict the life of such a pipe is evidently impossible. It is impracticable to prevent access of air and to turn the whole sewer into a septic tank, and the only reasonable course is to use some other kind

of pipe where the conditions mentioned are known or suspected, or to face the expense and trouble of occasional repairs.

Closely connected with corrosion of concrete by acids, actual or potential, is attack by electrolysis. All cement contains a small quantity of alkali, and this is an excellent electrolyte and will serve to convey such a current as may be straying from a lighting or power circuit. Instances have been recorded of destruction of concrete by such stray currents, and in this case, again, prediction of a probable life of the structure is clearly impossible. But stray currents are not in the same category, as wind and wave and earthquake, and their divagations should be prevented by proper insulation. To regard them as inevitable, like the rain, is not the attitude of mind of the electrical engineer, and it is to him that we must look for prevention. Suggestions to make the concrete waterproof, where there is a possibility of electrical leakage, are, however, well worth consideration, and in such cases, which should be rare, a sheath of some asphaltic material, such as is used for damp-proof courses, would be serviceable. But it cannot be too clearly said that this is the wrong principle to go on; it should not be necessary to protect concrete from stray currents, because those errant currents should be kept in their narrow channel.

The quality of cement for reinforced concrete must be at least as good as that for ordinary concrete, and, if possible, should be better. This is not because the latter should not be as near perfection as the maker can achieve, but because Portland cement for reinforced concrete is, as it were, a pioneer of progress, and what is a special brand for such purposes today will be the ordinary commercial article tomorrow. Turning to steel, as the other partner of the association, one may say that no better example of the advantage of that scientific direction which is now applied to Portland cement could be found than in the case of the steel, and it is significant that the metal, the more difficult of the two to manufacture, was being made of good and uniform quality before chemical principles were recognised and acted on in the manufacture of Portland cement. Thanks to the fact that for some forty years the regulation of the composition has been in the hands of the chemist, little is left to be desired in the modern commercial product. Of course,

cases have occurred, and will occur, of careless manufacture and inspection where brittle and inferior material has found its way into the work, but they are not numerous and only rank with such failures as arise in all structures. Good as modern mild steel is, it may be properly asked whether, in some cases at least, steel of a higher grade and greater tensile strength may be advantageously used, and I think that most of us will assent to this. This applies to ordinary structures and is, of course, obligatory for such buildings as safe deposits where the metal must not only have a good tensile strength, but be so hard as to be practically undrillable.

Turning now to reinforced concrete one may say that all the causes of attack, and consequently shortened life, which have been discussed under the heading of ordinary concrete, are valid equally with reinforced concrete, and, in addition, there are some other causes peculiar to reinforced concrete. In practice, the reinforcement consists of steel in some form, and is subject to the same corrosion as steel in other structures. By a very fortunate circumstance, cement is an alkaline substance and the metal, iron, in an alkaline medium does not rust. These comforting facts do not warrant the deduction that the steel reinforcement is immune from corrosion. That is true only if it is completely enclosed with concrete which is fully in contact with it and is free from fissures, a cogent reason for the use of concrete, for reinforced work, of a higher grade than that generally necessary. It is highly desirable that the concrete should not only be without fissures, but should be impervious. The advantages in preventing the percolation of any saline or corrosive substance are so great that the extra trouble and cost are well repaid.

The question of the probable life of reinforced concrete has formed one of the topics of a Committee of the Institution of Civil Engineers, of which the writer has the honour to be a member. At the request of the Committee I contributed a memorandum in February, 1910, which runs as follows:

"There are two matters affecting the stability and permanence of reinforced concrete which appear to me to be of special importance.

"The first is the risk of destruction of the cement in the

concrete, and of corrosion of the steel constituting the reinforcement; the second, the stresses due to expansion of the concrete and the steel, and to the difference in the value of the co-efficient of expansion of the two materials.

"The likelihood of the attack of the cement itself depends on the same conditions as determine whether ordinary concrete —unreinforced—is attacked, but as the thickness of the concrete over the reinforcement is usually small, it is evident that even more care must be bestowed on the avoidance of those conditions than in the case of concrete in large masses, where the surface of attack is relatively smaller. With regard to the steel, the risk of corrosion is primarily dependent on the continued access of water to the steel. Mere moisture left in the concrete after its preparation, or any stationary water, is of small consequence, because that water soon becomes saturated with lime and forms an alkaline medium excellently adapted for the preservation of steel. But continued access of water, involving anything approaching a flow of water, will remove this preservative lime, and cause the customary rusting of steel exposed to water. It follows that the concrete of armoured concrete must be practically impervious. This need for imperviousness has been recognised in practice by making the concrete rich in cement, but, as far as I know, no measurements of the permeability of the mixtures ordinarily employed have been published. Such experiments as I have carried out have been directed to a different end, and have been made with mixtures of cement and standard sand, and go to show that, though at first pervious, such mixtures soon become moderately watertight. Similar experiments with concrete composed of cement, sand, and small stones, such as is ordinarily used for reinforced concrete, would be useful. Impermeability of reinforced concrete, whilst desirable in all cases, becomes of great importance in structures immersed in fresh water, and imperative for those in sea water.

"The co-efficient of expansion by heat of mild steel is well known, the figures generally accepted being 0.0000069 per degree Fahrenheit. That of concrete varies somewhat with the composition, but for a 1:2:4 mixture is usually taken at 0.0000055 per degree F. For a mixture of 1 of Portland cement to 1 of sand, I have found 0.0000053 per degree F. wet and

0.0000073 per degree F. dry. These values are so similar that the stresses resulting from their difference are small. But the stresses due to the two materials regarded as a whole in a structure hindered from expansion may be considerable, and should certainly be computed in cases where the structure is exposed to a large range of temperature. Further, there are no data (so far as I have been able to ascertain) concerning the alteration of length of concrete which takes place by alteration of wetness and dryness. If this, as seems probable, is considerable, serious stresses may arise, both in a structure not free to expand and between the concrete and its reinforcement. Experimental investigation of this question appears to me to be desirable.''

After five years, I think that this brief summary states not unfairly the conditions which influence the stability of reinforced concrete, and indicates the directions in which precautions should be taken. Since that time many experiments have been made, but they are not so conclusive as might be wished. At the instance of the Committee of the Institution of Civil Engineers, referred to above, experiments were made at the National Physical Laboratory with a view to determine the rate of percolation and the alteration of strength, but the results are irregular and will not serve as a useful guide. It is highly desirable that a full set of tests should be made to obtain reliable data on these two points, viz., rate of percolation and alteration of strength of reinforced concrete; and, in both cases, the test pieces must be prepared by an operative accustomed to such work, and carrying it out as it would be carried out in practice under proper supervision. As far as I know, no reliable data obtained by experiment, as distinct from observation, are extant, showing whether reinforcing steel will corrode; and here again, a full set of tests should be made. Casual observations of the condition of the metal in reinforced concrete which has been exposed to severe natural conditions are of the utmost value, but we want something more than that. We want definite facts which will tell us what is the prospect of life of reinforced concrete properly and carefully, but not meticulously, made, when it is exposed to the most drastic conditions which it will be called upon to endure in practice; and one of the governing factors is the non-rusting of the steel. *A priori*, there should be

no rusting, but argument *a priori* is the most dangerous form of nonsense I know of. In matters of this material importance, it is not sufficient to come to a conclusion on general principles alone, but these must be used in conjunction with experimental data as accurately obtained and obtained over as long a period as practical requirements allow. And here let me plead for long time tests. It is true that they may be useless and obsolete before their term is out, 20 years, 50 years it may be, not much in the life of a structure, but the trouble and cost of making them is trifling, and sometimes their results are priceless. Let us build for posterity in this matter; it is easy for them to discard our juvenile ideas, but now and then they may find something good like the "Principia" or an older book. With the knowledge of this date, it seems fairly certain that little fear need be felt of steel reinforcement rusting when well embedded in good non-pervious concrete of adequate thickness, even when the structure is exposed to seawater or other saline solutions, but the case is altered when the concrete is exposed to electrolysis. As has been mentioned above, cases have been observed of the destruction of ordinary concrete by electrolysis, and the risk of injury to that is small compared with the likelihood of destruction of reinforced concrete by the same cause. The advantage of an alkaline medium may disappear, and the steel reinforcement serving as a positive electrode may be attacked by all the negative ions of the electrolyte. Corrosion will be rapid, and the stresses exerted by iron rusting are known to be large, though they have never been computed. It must be remembered that it is not necessary for there to be a direct electrical leak from the inside to the outside of the concrete. Wherever the current flows there must be a drop of potential, and as the joints between the metallic members are electrically poor, it is certain that at all those points corrosion must occur. In a modern structure honeycombed by electric leads, most serious results may occur from such unsuspected cause, and the mischief may be wrought secretly and effectively, quite nullifying any reasonable presumption of the life of the structure.

There is another fact which tends to limit the life of a structure made of reinforced concrete. One of the great advantages of this material is its homogeneity. A properly designed and

made structure is as much of a piece as if it were a casting, and, like a casting, experiences internal stresses. These can be minimized or provided for by careful design, but there is no process equivalent to annealing by which they can be removed. Instances are on record where cracking has happened in long continuous lengths or in large thin walls or panels, which, assuming material and work to be free from fault, must have been caused by internal stresses. There is always some stress, and the amount may be increased by part of the structure being wet and another part dry, and it is just on this point that very little exact information is available. Because, by a happy accident, the co-efficients of expansion of steel and concrete are nearly identical, it has been too hastily assumed that stresses between the two are negligible, the fact being overlooked that wetting steel has no effect on its size, and wetting concrete has a well marked effect. It must not be thought for one minute that I am an alarmist, but it is right to point out a cause of trouble which may escape consideration, if a generalisation, in the main good and sound, is accepted without reservation. Thorough investigation, using large test pieces over a long period and under perfectly determinate conditions, would be of the utmost value, and would afford us data superseding the somewhat casual observations on which too much reliance has hitherto been placed.

That this knowledge is of much more than academic importance will be admitted when the construction of dams in reinforced concrete is considered. In a dam, every element of destruction of the kind which has been discussed must be studied and prevented. The concrete must be watertight, for any percolation through pores or cracks will be much more injurious than a similar leakage through an ordinary concrete or masonry dam. The very core of the structure will be attacked, and its ruin is only a question of time. The fact that a dam is a monolith, and may be a huge one, with a crest which is slim and delicate compared with the base, and that it lies, as it were, between wind and water, wet on one side, dry on the other, with a fluctuating height of wetness and a varying load, enforces the absolute necessity of knowing precisely what internal stresses must be met.

The chief causes which go to shorten the life of structures

made of concrete and reinforced concrete having been reviewed, it remains to consider how far these causes are effective, and, if possible, to fix a fair average term of life for such structures.

Of all the causes of destruction, by far the most important is corrosion by saline solutions, and it is the most insidious, as the structure exposed to the action of the solution—whether a harbour or a sewer—may be covered by the attacking liquid, and difficult to examine. For such structures, impermeability is imperative. Reliance on any form of silting up, taking up, covering by organic growths, is mere lazy folly. The material must be free from interspaces which are not microscopic and disconnected. Anything approaching a channel is undoubtedly mischievous, and may be fatal. This axiom has been arrived at painfully and with heavy cost in the hard school of experience before reinforced concrete was thought of, and is doubly axiomatic—if one may be pardoned the term—when the concrete has in its heart a more sensitive core, protected, it may be, by a layer only two or three inches in thickness. At the risk of being tedious, I reiterate that the permanence of concrete depends on its imperviousness, and that any condition which limits this limits its life almost *pari passu.*

All other causes which tend towards destruction sink into insignificance beside this, but for all that they must not be ignored, and, without attempting to arrange them in order, I am inclined to think the next worst, closely approaching the severity of attack of saline solutions, is the injury caused by aggregates of the class of coke breeze, containing sulphates or potential sulphates. From the very nature of the material, and from the use to which it is put, namely, to make light floors, ceilings and partition walls, it is clear that it cannot be impervious, and it follows that whatever water reaches one of its surfaces will speedily make its way to the interior. Where water can go, air can follow, and the assumption that sulphides are fairly harmless falls to the ground, because they are in the most favourable condition to become sulphates, and the fate of the structure is then settled.

Next in order of sinister magnitude is the injury caused by electrolysis. It is true that the cases recorded are at present few, but it must be remembered that the

transmission of large currents at high pressures is a comparatively modern thing, that large structures of reinforced concrete are comparatively modern things, and the most progressive spirits cannot hope to see buildings only five years old fall like the walls of Jericho. It may soothe their natural impatience to reflect that although the structures and the electrical power are fairly new, yet human blundering is fairly ancient and to be relied on, and on that ground alone, it is reasonable to suppose that failures induced or exaggerated by electrolytic action will become fairly common, particularly, when there is a steel core to attack and an electrolyte additional to those alkali salts which naturally occur in any normal cement.

It has been necessary to indicate all those causes of injury for destruction which are to be reckoned with as affecting the life of concrete structures made with modern cement, and the impression conveyed may be that all such structures are so liable to decay as to be almost ephemeral. But this paper is not a jeremiad. I have sought to state the very worst against the prospect of life of concrete and its younger brother; I have purposely taken the role of the devil's advocate, and have striven to seek out and blazon all the faults and vices proper to cement as we know it. And what does this assault amount to? In the first place, that there exist causes of destruction, internal and external, which, if uncontrolled, will certainly destroy any structure, even when its design is impeccable, and that its life is at the mercy of these causes, and though its death may be lingering, it is certain. In the next, that all such causes, except extreme external violence, can be controlled, and their effect nullified by knowledge, care, and skill exercised in the directions mentioned and discussed above; and, as a necessary result, by the practical elimination of nearly all these attacking forces, security and something like permanence will be attained.

I began this paper by saying that there is no such thing as a permanent structure, and I end by stating that in my opinion a pyramid, a bridge, an aqueduct, a harbour, a coliseum or a cathedral made in concrete or ferro-concrete, well and honestly enough, will vie in life with the mighty structures of antiquity which are left to us, their limitation of life lying not in themselves, but in the changing needs of human life.

DISCUSSION

Mr. H. B. Muckleston*, M. Am. Soc. C. E., called attention to the fact that this paper does not discuss dense concrete, nor the effect of alkali. Mr. Muckleston.

In irrigating ditches, alkali water will sometimes eat out the concrete in one year, even though it be first-class work. Magnesium sulphate affects concrete everywhere.

Mr. Chas. W. Baker**, Mem. Am. Soc. M. E., said that concrete exposed to sea water, where frost conditions obtain, will not last. The salt water freezes in the zone between high and low tides. In northern localities, the permanence of concrete depends on its impermeability. Mr. Baker.

Progress along these lines depends on developing processes for making dense concrete.

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Paper No. 103

VOLUME CHANGES IN CONCRETE.

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INTRODUCTORY.

This paper aims to review our present knowledge of the volume changes which take place in concrete. It is assumed that the mineral aggregate is in every case stable and inert and that all changes are due to the cement itself. It is further assumed that the cement is normal Portland cement which passes standard specifications. The influence of unusual chemical agents is also excluded, so that the discussion resolves itself almost entirely into a consideration of the influences exerted by temperature and by water.

CHANGES DURING THE INITIAL HARDENING PROCESS.

When Portland cement reacts with water the important change is the development of a colloidal cement. This theory was first set forth by Michaëlis in 1893(1) and developed by him in later papers. It is usually believed that there is no appreciable change in volume during the hardening process, but Le Chatelier(9), and, later, Kühl(19) showed that there is an actual decrease in the absolute volume. Their experiments were made in glass flasks whose bulbs were filled with a wet paste of cement and whose stems were filled to a given mark with clear water. In one of Kühl's experiments 250 grams of cement mixed with 200 cc. of water showed a shrinkage after three hours of 1.4 cc., after twenty-four hours of 5.0 cc., after

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(1) Numbers in parentheses refer to the appended bibliography.

seven days, of 10.6 cc. and after twenty-eight days, of 13.6 cc. All cements, even those which contained free lime and ultimately expanded, showed this phenomenon of contraction, the decrease in volume persisting even up to the time when the expanding cements burst their enclosing vessels through the pressure exerted upon their walls. Le Chatelier makes the distinction between the absolute volume which decreases, and the apparent volume which increases.

The apparent anomaly may be explained by a consideration of the phenomena attending the hardening process. Cement reacts with water to form a colloid of greater volume than the cement but less than the combined volumes of the cement plus the water. After the cement has attained its initial set and become relatively rigid, the process of hardening still goes on through diffusion of water to fresh surfaces of cement, as indicated by the falling water level in the neck of the flask. The newly formed colloid has larger volume than the cement, therefore, pressure is exerted on the limiting surfaces. If the hardened cement were plastic, the pressure in the experimental flask would be relieved upward and the surface of the water in the neck of the flask might rise. Since, however, the hardened mass is relatively rigid, the pressure is exerted practically equally in all directions and the glass must crack.

VOLUME CHANGES IN CONCRETE CONTINUALLY IMMERSED IN WATER.

The expansion occurring in neat cement bars continuously stored under water was first measured by Bauschinger(2) in 1879. Subsequent measurements have been made by Schumann(4), Tomëi(5), Considère(7), Gary(8), Campbell and White(14), White(15), Goldbeck(16), Graf(18), Jahn(20), Rudeloff and Sieglerschmidt(21), The Bureau of Standards(26, 28) and Spackman(31). The results of all these investigators show that neat cement, and to a lesser extent concrete, expand when kept continuously in water. The results of those authors who have made tests lasting more than one year are summarized in Table I. Some individual tests by the author of this paper are shown graphically in Figures 2 and 3.

Table I. Percentage Change in Length of Bars of Neat Cement and Cement Mortar When Immersed Continuously in Water.

Author	Neat Cement	1:3 Cement and Sand	1:4 Cement and Sand	Duration
Schumann	+0.150	+0.027		5 years
Gary	+0.092	+0.020		2 years
Campbell and White,				
Fresh cements	+0.167			5 years
Aged cements	+0.089			5 years
White	+0.111	+0.025		3 years
Graf			+0.018	6 years

The variations in the figures found by different observers are undoubtedly due not only to variations in the cement but also to the differing quantities of water used in gauging, the amount of tamping, the quality of the sand and the conditions surrounding the initial measurement. The figures are, however, fairly concordant. The action of the water on the cement evidently continues until the colloid developed becomes so dense that no more water can diffuse through it. This condition probably prevails in neat cement blocks after about twelve months since no further expansion is evident after that time. With mortars and concrete the volume of voids to be filled is larger and the observed expansion is, therefore, less. With very lean mixtures it may happen that the colloid becomes fully developed through hydration of the cement without the voids becoming filled with colloid. Such concrete will, of course, show no expansion and must remain permanently porous.

The expansion of concrete which remains continuously in water is relatively slight and its influence probably beneficial since it fills the voids and puts the concrete in slight compression. It is known that concrete dams, which are initially porous enough to allow water to seep through, become tight after some months' service. This change is probably due to the progressive development of the colloid.

VOLUME CHANGES IN CONCRETE KEPT CONTINUOUSLY IN AIR.

The changes in volume of concrete kept continuously in air at room temperature have been studied by Bauschinger(2), Schumann(3), Tomëi(5), Considère(7, 30), Campbell

and White[14], White[15], Graf[18], Jahn[20], Rudeloff and Sieglerschmidt[21], The Bureau of Standards[26, 28] and Spackman[31]. Most of these tests were of only a few weeks' duration but all agree in showing marked contraction. The results of tests which have extended more than six months are summarized in Table II.

Table II. Percentage Change in Length of Bars of Neat Cement and Cement Mortar When Stored Continuously in Air.

Author	Neat Cement	1:3 Cement and Sand	1:4 Cement and Sand	Duration
Campbell and White	—0.390			5 years
White	—0.321	—0.072		4 years
Graf	—0.242	—0.111*	—0.043	2 years
Considère			—0.037	16 months
Bureau of Standards	—0.290			6 months

This shrinkage in air is of much greater magnitude than is the expansion in water and is more often productive of harmful results since the concrete is placed in tension. Mixtures rich in cement are much more apt to crack than lean mixtures and especial attention must be paid to the design if cracks are to be avoided.

If the concrete is not reinforced and its ends are rigidly restrained it will almost necessarily crack. This is illustrated in Figure 1, which shows the cracks in a series of bars made of a mixture of 1 cement and 2 sand and moulded in heavy cast iron frames designed to grip the ends of the bar firmly and yet not touch the central portion of the bar. The ends of the bar were gripped by teeth projecting from the frame and also by threaded bolts embedded for six inches at each end. There was thus left a free bar of concrete thirty-two inches long and four inches square, which was rigidly restrained at its ends. Each bar carried imbedded in it two brass plugs twenty inches apart for measurement of change in length. Six of these restrained bars and six unrestrained companion bars were cast at the same time. The forms were stripped after twenty-four hours and the bars kept damp for varying periods. The restrained bar A and its companion bar were placed in the air

* = 1:2 Cement and Sand.

of the room and allowed to start the drying process after twenty-four hours. In eight days a faint crack appeared which, in its later development, is plainly visible in Figure 1. When the crack first appeared, the shrinkage of the restrained bar was 0.015 per cent and that of the unrestrained bar 0.023 per cent. As the shrinkage became greater the crack became wider until the measured length finally became fairly constant with a contraction of 0.135 per cent on the twenty-inch portion of the restrained bar and 0.185 per cent on a similar length of the unrestrained companion bar. The other bars were kept damp for longer periods of time, bar C for one month and bar D for six months. They stayed in perfect condition as long as

Fig. 1. Cracks Caused by Drying 1:2 Mortar Bars Cast with Restrained Ends.

they were damp, but within ten days after exposure to the air each restrained bar developed a crack which widened as the bar became drier until the values shown were substantially those given above for bar A.

VOLUME CHANGES IN NEAT CEMENT WHEN ALTERNATELY WETTED AND DRIED.

It has apparently been generally assumed that, whatever the nature of the reactions taking place during the hardening of cement, the cement after hardening becomes stable and constant in volume. The fact that concrete undergoes relatively large changes in volume with changing moisture content has only recently been investigated. Keller (6), in 1894, in his in-

vestigations of the thermal expansion of concrete, noted that the moisture of the air affected his results and that, when the bars on which he was working were immersed in water for two days, they grew perceptibly longer. Binnie([13]), in 1905, also noted that the humidity of the air influenced the length of concrete. Systematic investigations of these changes have been published by White([15]), in 1911, Jesser([23]), Rudeloff and Sieglerschmidt([21]), in 1913, and White([29]), in 1914. These authors all agree that cement and concrete expand whenever placed in water and contract whenever dried in air. Jesser calls special attention to the influence of the moisture of the air. The tests of Jesser and of Rudeloff and Sieglerschmidt were made on green concrete and did not extend for a period of more than six months.

In previous papers the author has reported tests on alternate wetting and drying of cement bars for a period of six years after they were made up, and also tests on bars cut from concrete which had been in place twenty years. There has not been a single exception to the rule that concrete expands when placed in water and contracts when placed in air. Curves, representing five different brands, giving the changes in length of bars of neat cement, are shown in Figure 2. Bars 131 E, G and I represent three different brands of commercial cements treated under the same conditions. It will be noted that they all expanded during the first year in water but remained almost entirely constant in volume during the next two years. On being dried they contracted, the change in length being very uniformly 0.15 per cent, and on being immersed in water expanded again to slightly more than their initial length. These changes have repeated themselves with each cycle but in general these bars are becoming longer and the change in length between the dry and wet state on the last alternation averaged 0.19 per cent instead of 0.15 per cent as in the first cycle. It is evident that more colloid is being developed with each long immersion in water and that as a consequence the maximum length of the bars and also their fluctuation in length is increasing.

The colloid, which has once developed, responds much more rapidly to water than does the original unhydrated cement. Although it took twelve months' continuous immersion

to hydrate for the first time all the cement of these bars which was accessible to water, the re-hydration of these same bars after two months in air proceeded so rapidly that 90 per cent of the total expansion, or about 0.13 per cent, was re-

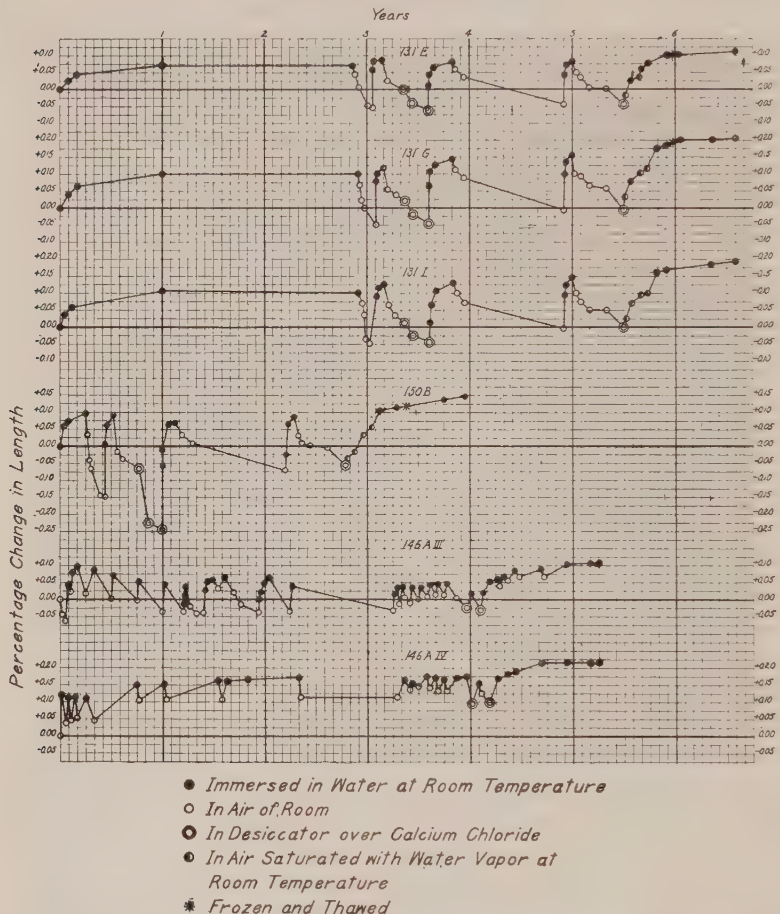

Fig. 2. Changes in Length of Bars of Neat Cement when Alternately Wetted and Dried.

corded in the first twenty-four hours. On another occasion after the bars had been in air over a year, the expansion, after immersion in water for four hours, was about twenty-five per cent of the total and after twenty-four hours, was sixty per

cent of the total expansion caused by prolonged immersion in water. The shrinkage in air takes place much more gradually.

The marked influence of atmospheric humidity is shown on the last hydration of these same bars, 131 E, G and I in Figure 2. These bars increased in length more than 0.10 per cent on being taken from dry air and placed for two months in air saturated with moisture. These experiments on bars six years old confirm the work of Jesser on bars only a few weeks old.

The three bars of the 131 series, shown in Figure 2, were all kept in water continuously for the first three years of their lives. Bar 150 B is added as an illustration of the behavior of a neat cement kept in water only three months and subjected to rather long periods in dry air alternating with shorter immersions in water. The general type of curve is the same as that of the preceding bars but the total change in length, either because of a peculiarity of the cement or because of its treatment, is unusually high and reaches 0.40 per cent.

All of the volume changes in these neat cements are due to changes in moisture, the temperature of the room where the bars were stored varying hardly 10° C. from one year's end to the other. Since a change in temperature of 10° C. amounts to only 0.01 per cent, the error is negligible. Although the volume changes recorded here were not affected by temperature changes, it should be stated, for completeness of the record, that bars 131 E, 131 G and 150 B were on one occasion, as indicated by the asterisks in Figure 2, purposely subjected to freezing tests. The bars did not suffer any permanent change in volume in these tests which are described later.

The effect of more frequent alternations in moisture content of neat cement bars is shown by curves 146 A III and A IV of Figure 2, where the linear changes in two bars of neat cement made up to be duplicates of each other are shown. Bar 146 A III was kept in air for relatively long periods, and for shorter periods in water. As a consequence its mean length during the first four years was not much different from its initial length. Bar 146 A IV was, on the contrary, kept wet for relatively long periods, so that its mean length during the same period was about +0.15 per cent. Early in the fourth year an attempt was made to bring 146 A III up to the length

of its duplicate by changing the schedule of its alternations so as to give it longer periods in water. Its growth is very marked but its duplicate has kept pace with it, so that both are now longer than ever before in their history.

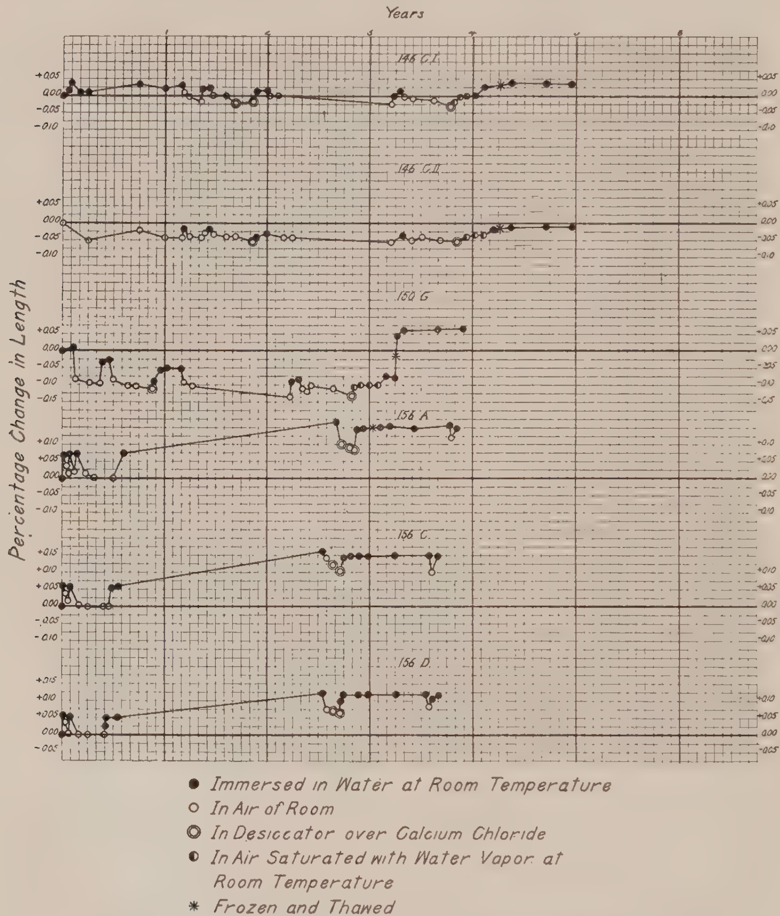

Fig. 3. Changes in Length of Concrete when Alternately Wetted and Dried.

CHANGES IN CONCRETE WHEN ALTERNATELY WETTED AND DRIED.

The changes in concrete, when alternately wetted and dried, are represented by the curves in Figure 3. The bars numbered 146 and 150 are made from one part of cement and

three of standard Ottawa sand and the behavior of the neat cement is shown under the corresponding number in Figure 2. It would naturally be expected that bars made from the coarse and evenly sized Ottawa sand with its large percentage of voids would show even less change than its diminished cement ratio would require, yet the changes here shown are all higher than would be expected and amount to more than 0.05 per cent on the last alternation, which is equivalent to a temperature change of 50° C. These bars were kept at practically constant temperature from the time they were made up until the point indicated by the asterisks when they were frozen, as explained in the discussion of neat cement bars. The 146 bars were unaffected in length after the freezing process, but bar 150 G suffered decided expansion. The matter is discussed further in the section on freezing tests.

The bars of Series 156 of Figure 2 were sawed by the author from blocks taken from cement sidewalks which had been in service about twenty years. Bar 156 A was sawed from a piece of the top coat of a walk which had spontaneously separated from its base. It was hard, strong and apparently well mixed. Bars 156 C and D were taken from a sidewalk in good condition, 156 C being sawed from the top coat and 156 D from the base immediately adjacent to 156 C. These bars do not show any lessened variation in length with alternate wetting and drying in spite of their twenty years' exposure to the weather. On the contrary the total elongations of the two samples of top coat from different walks are almost as great as those shown by neat cement, while the base with its much smaller proportion of cement shows two thirds as much expansion. A study of the changes in weight and specific gravity of these bars, which will be found in the original paper, confirms the following explanation of these changes.

The colloid, which results from the hydration of cement, expands and contracts with changes in moisture. If the surroundings are relatively dry, or if they are dry for long periods and wet for short ones, the concrete remains shrunken. If, on the other hand, the environment is such that the concrete is saturated with water frequently, it expands. With each successive expansion opportunity is allowed for water to reach

clinker particles hitherto unacted upon, so that under proper conditions the concrete may grow in length even though it may already be many years old.

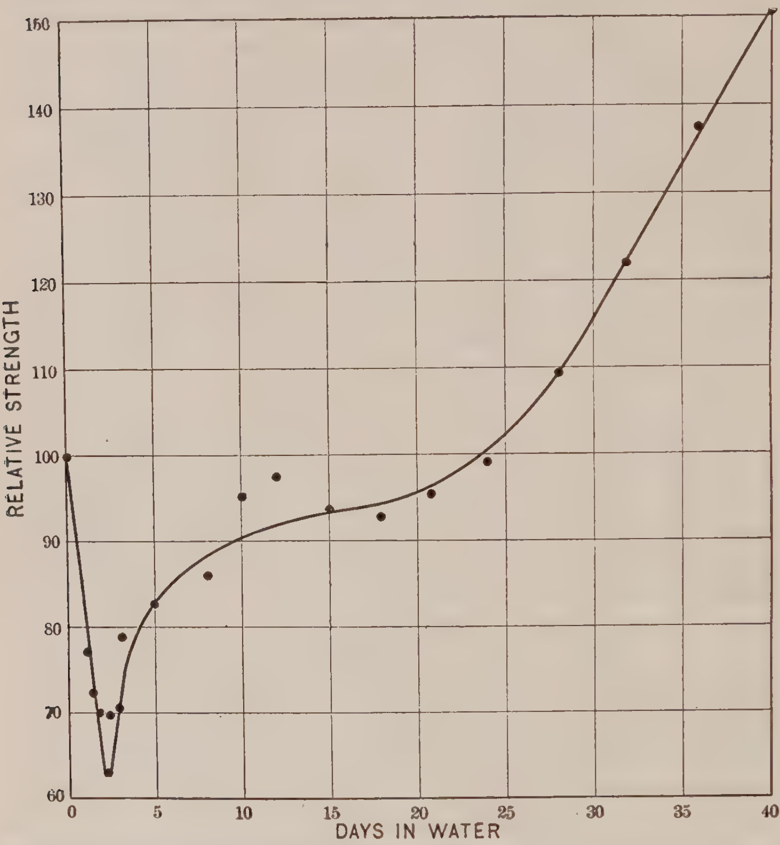

Fig. 4. Percentage Variation in the Crushing Strength of Concrete Cylinders after Immersion in Water for Various Periods.
(From Van Ornum, Trans. Am. Soc. C. E., Vol. 77, p. 438.)

EFFECT OF MOISTURE ON THE STRENGTH OF CONCRETE.

It seems to have been recognized vaguely by a number of engineers that some relation existed between the amount of moisture in concrete and its crushing strength. The most extensive investigation is by Van Ornum(32), who made two hundred and forty cylinders of 1:2:4 sand and gravel concrete, removed them from the moulds when two days old and tested

them in compression when six weeks old. The cylinders were all placed in air when removed from the moulds, but at regular intervals portions of them were immersed in water and allowed to remain there till the tests were made. When they were finally tested all were six weeks old, but some had been in air continuously, some had been forty days in water and others had been during the first portion of that period in air and during the latter period in water. The curve of the relative strength of these cylinders is given in Figure 4 where the strength of the air-dried bars is rated at 100. It will be observed that, with short immersions in water, the strength falls off rapidly and reaches its minimum after two days in water, when the strength is less than two-thirds that of the cylinders which had remained in air. The strength rises rapidly with longer immersion in water and after twenty-five days in water becomes decidedly greater than that attained by bars stored in air.

This phenomenon may be readily explained in the light of the volume changes caused by moisture. Water soaking into concrete expands the colloid rapidly. There will be a strain between the expanded shell and the unexpanded core which will weaken the concrete. The removal of this strain will require a longer time than its inception, for the developed colloid on the surface will allow the water to diffuse more and more slowly to the center. The cylinders used by Van Ornum were eight inches in diameter and sixteen inches high. Had they been twice as thick it would have required a longer time to reach the point of minimum strength and a disproportionately longer time to again reach the greater strength shown by bars remaining in air.

Concrete which has thoroughly soaked in water should show the same decrease in strength when dried rapidly, as the outer shell tends to shrink, while the inner core is still swollen. This has been shown by Van Ornum(12) to hold true for the neat briquettes used in cement testing. It apparently did not hold true for briquettes of 1 cement to 3 sand, where the equalization of the water content in the small cross section probably proceeded rapidly enough to prevent any unfavorable effect from becoming evident.

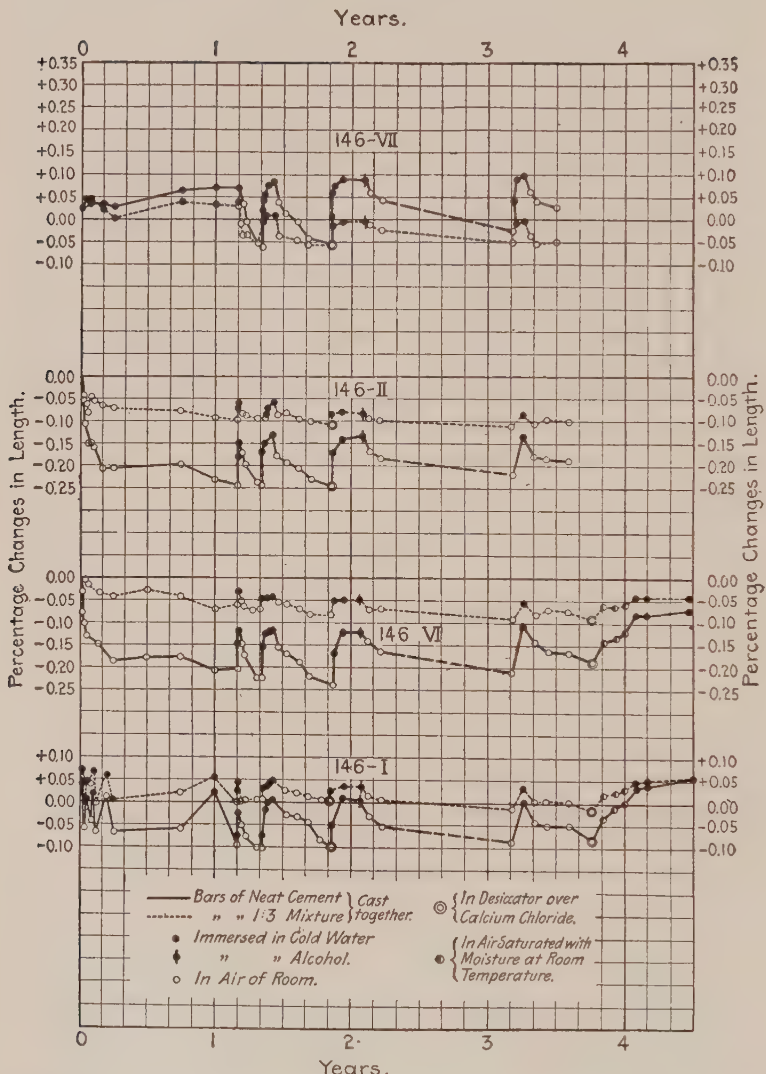

Fig. 5. Changes in Length of Compound Bars when Alternately Wetted and Dried at Room Temperature.

DIFFERENTIAL EXPANSION OF MIXTURES OF DIFFERENT RICHNESS.

The relative rate and extent of volume changes in superimposed layers of different richness in cement have an important bearing on the life of concrete pavements and walks. The behavior of some compound bars of neat cement, cast integrally with a bar of mortar made from 1 part cement to 3 of sand, is given in Figure 5. It will be noted that the neat cement changes more rapidly and to a greater degree than does the sand mortar, so that there must be a reversal of the strains with every large change in moisture content. This explains why the top coat so frequently splits from the base of old cement sidewalks.

COEFFICIENT OF THERMAL EXPANSION OF CONCRETE.

Shitkewitsch(24) reviews very fully the data regarding the coefficient of expansion of various cement mortars and other building materials. His data regarding cement and concrete are quoted in full in Table III, where are also given his average figures for the other building materials. The figures of Keller(6) and the recent data of Rudeloff and Sieglerschmidt(21) have been added.

The average of all the figures for concrete agrees very well with the usually accepted figure 0.00001 for 1°C. or 0.0000055 for 1°F. Rudeloff and Sieglerschmidt state that the coefficient of concrete increases with its age for the first several months of its life but that for practical purposes the above value is sufficiently accurate. Concrete, which has been very recently made, offers some exception to the above figures. They report that samples hardened under water for four and nine days showed contraction instead of expansion when heated to 50°C. All samples which had been in air expanded when heated, as did also those kept more than nine days in water.

EFFECT OF HIGH TEMPERATURES.

The most important work on this subject has been done by Norton(23), who heated cubes of concrete made from 1 cement, 2 sand and 5 stone in an electric furnace until they had come completely to equilibrium and then measured the changes. His average results are given in Table IV.

Table III. Coefficient of Thermal Expansion of Cement and Other Building Material.

Material	Linear Coefficient of Expansion For 100° C.	For 100° F.	Authority
Neat Cement	0.00122	0.00068	Meier.
" "	0.00145	0.00081	Meier.
" "	0.00143	0.00079	Adie.
" "	0.00107	0.00059	Bouniceau.
" "	0.00137	0.00076	Hyatt.
" "	0.00140	0.00078	Hyatt.
" "	0.00126	0.00070	Keller.
Mortar of 1 cement, 3 sand	0.00118	0.00066	Bouniceau.
" " 1 cement, 1 sand	0.00110	0.00061	Keller.
" " 1 cement, 2 sand	0.00101	0.00056	"
" " 1 cement, 4 sand	0.00104	0.00058	"
" " 1 cement, 6 sand	0.00092	0.00051	"
" " 1 cement, 8 sand	0.00095	0.00053	"
Concrete of 1 cement, 2 sand and 4 Bedford limestone	0.00098	0.00055	Pence.
Concrete of 1 cement, 2 sand and 4 Kankakee limestone	0.00100	0.00056	Pence.
Granite (average)	0.00081	0.00045	Bouniceau, Adie, Hurst, Dana.
Limestone (average)	0.00091	0.00051	Bouniceau, Dana, Pence.
Marble (average)	0.00076	0.00042	Bouniceau, Adie, Ganet.
Sandstone (average)	0.00124	0.00069	Haswell, Ganot, Dana, Thurston, Adie.

Table IV. Mean Coefficient of Expansion of Concrete Heated to High Temperatures.

Temperature Deg. Fahr.	Deg. C.	Value of B in expansion $l_t = l_o (1 + Bt)$
72 to 360	162 to 680	0.0000045 to 0.0000060
72 to 750	162 to 1382	0.0000050 to 0.0000060
72 to 1090	162 to 1994	0.0000045 to 0.0000050
72 to 1600	162 to 2912	0.0000035 to 0.0000042

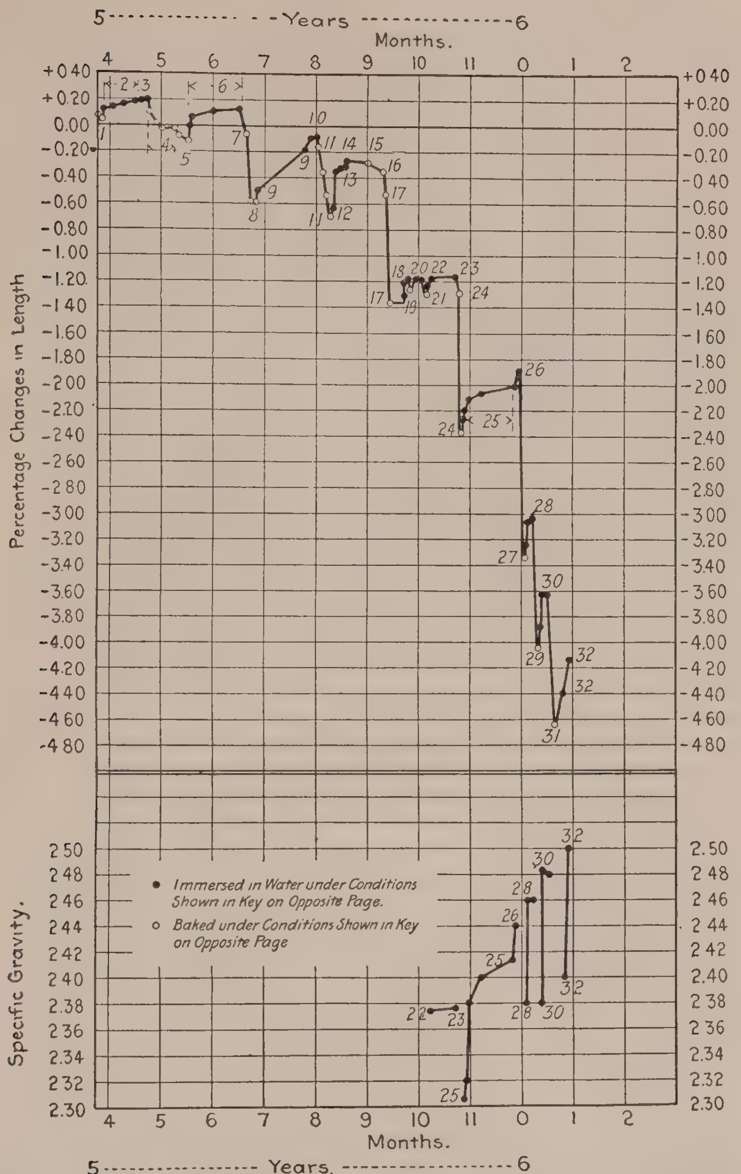

Fig. 6. Changes in Length and Specific Gravity of a Bar of Neat Cement when Alternately Boiled and Baked.

Key to Fig. 6 and Tabular Summary of Changes in Length and Density of a Bar of Neat Cement When Alternately Boiled and Baked.

All Measurements Made at Room Temperature.

Point No.	Description of Test.	Percentage Change in Length Referred to Initial Measurements of Freshly Made Bar.	Specific Gravity	
			Beginning.	End.
	Bar air-dried and 5 years 3 months old at beginning of test ...	+0 091	.....	.....
1	Heated 2 hours at 100 to 105° C	+0 062	.....	.....
2	In cold water up to 21 days.	+0 171	....	.....
3	Boiled for 3 periods with total of 52 hours...........	+0 208	.....	.....
4	Baked at 85° C. for total of 13 days....	−0.011	.....	.....
5	Baked at 130 to 150° C. for 4 periods of 8 hours each...	−0.118	.. ..	.....
6	In cold water up to 30 days............................	+0.139	..	.....
7	Baked at 90° C. for 24 hours..	−0.053	.	.....
8	Baked at 250 to 300° C. for 2 periods of 20 hours each.......	−0 590	....	.....
9	In cold water up to 30 days....	−0 190	.. ..	.....
10	Boiled 2 periods of 24 hours each..	−0.091	... .	.....
11	Baked for 24 hours each at 85, 150, 250, 340° C............	−0.708	.....	.....
12	In cold water in vacuum 5 hours............	−0.673	. . .	.....
13	Boiled 3 periods of 24 hours each.........................	−0.307	.....	.....
14	Boiled in water under 75 lb. pressure 8 hours	−0.287	.....	.....
15	In cold water 2 weeks..	−0 287	.....	.....
16	Dried in air 1 week and at 85° C. for 48 hours...............	−0.342	.....	...
17	Baked at 110° C. for 24 hours and at 600° C. for 18 hours....	−1 358	.....	.....
18	In cold water in vacuum 1 hour and boiled 2 periods of 24 hours each	−1 188	.....	....
19	Baked at 85° C. for 12 hours.	−1.260	.....	...
20	Boiled 2 periods of 3 days each	−1 162	....	...
21	Baked at 101° C. for 24 hours..	−1.301	....	....
22	In cold water in vacuum 1 hour and boiled 36 hours.	−1 165	2.375	.....
23	In cold water for 23 days	−1 157	.....	2.377
24	Baked 24 hours at 101° C. and 24 hours at 475° C..	−2.388	.....	.....
25	In cold water in vacuum for 1 hour and then in cold water 1 month	−2.015	2.305	2 412
26	Boiled 48 hours	−1 888	.....	2 441
27	Baked 24 hours at 85° C. and 16 hours at 500° C	−3.343	. . .	....
28	In cold water in vacuum ½ hour and boiled for 2 periods of 18 and 36 hours	−3.037	2.381	2.460
29	Dried 24 hours at 85° C. and baked 18 hours at 550° C . .	−4 074	.. .	.....
30	In cold water in vacuum ½ hour and boiled 2 periods of 36 hours each.....	−3 641	2.379	2.481
31	Dried 24 hours at 85° C. and baked to 600° C. for 18 hours .	−4.613	..	. ..
32	In cold water in vacuum ½ hour and boiled 36 hours. . . .	−4 166	2.401	2 501

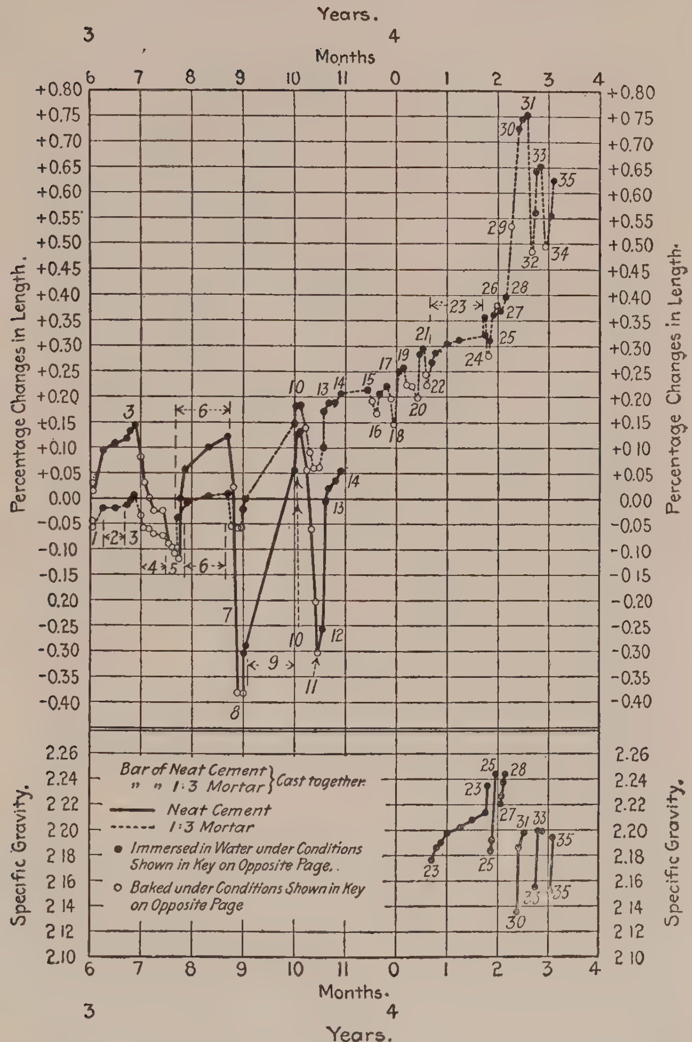

Fig. 7. Changes in Length of a Compound Bar when Alternately Boiled and Baked.

Key to Fig. 7 and Tabular Summary of Changes in Length and Density of a Compound Bar of Neat Cement and 1:3 Sand Mortar When Alternately Boiled and Baked.

All Measurements Made at Room Temperature.

Point No.	Description of Test.	Percentage Change in Length Referred to Initial Measurement of Freshly Made Bar.		Specific Gravity.	
		1:3 Mortar.	Neat.	Beginning.	End.
	Bar air-dried and 3 years 6 months old at beginning of test	−0.048	+0.027	.....	.....
1	Baked 2 hours at 100 to 105° C.	−0.062	+0.013	.....	.....
2	In cold water up to 21 days	−0.014	+0.120	.....	.....
3	Boiled 3 periods with total of 52 hours	+0.007	+0.138	.....	.....
4	Baked at 85° C. for total of 13 days	−0.074	−0.025	.....	....
5	Baked at 130 to 150° C. for 5 periods of 8 hours each	−0.084	−0.116	.....	.....
6	In cold water up to 30 days	+0.011	+0.124	.....	....
7	Baked at 90° C. for 24 hours	−0.052	−0.022	.....	....
8	Baked at 250 to 300° C. for 2 periods of 24 hours each. (At the end of this treatment the bar of neat cement was cracked completely through near the middle and showed other hair cracks.)	−0.059	−0.373	.....	....
9	In cold water up to 30 days	+0.145	+0.055	.....	.....
10	Boiled for 2 periods of 24 hours each	+0.180	+0.131	.....	.....
11	Baked for 24 hours each at 85, 150, 250 and 340° C.	+0.063	−0.303	.....	....
12	In cold water in vacuum 5 hours	+0.098	−0.255	.....	....
13	Boiled 3 periods of 24 hours each	+0.187	+0.035	.....	.....
14	Boiled 8 hours in water under 75 lb. pressure. (The neat portion which had cracked through in several places was sawed off and discarded, leaving the 1 : 3 sand portion intact.)	+0.208	+0.054	.....	.....
15	In cold water for 2 weeks	+0.208	....	.....	.....
16	Baked for 24 hours each at 85 and 110° C.	+0.169	....	.....	.....
17	In cold water in vacuum 24 hours, then boiled 24 hours	+0.220	.....	.....	.....
18	Baked 24 hours at 85° C. and 24 hours at 450° C.	+0.140	.....	.....	....
19	Boiled 2 periods of 24 hours each	+0.253	.....	.....	.....
20	Baked for 24 hours each at 85, 100 and 450° C.	+0.199	.....	.....	.....
21	Boiled for 2 periods of 24 hours each	+0.297	.....	.....	.....
22	Baked 24 hours at 100° C. and 12 hours at 450° C.	+0.220	.....	......	.....
23	In cold water up to 43 days and then boiled 24 hours	+0.358	.....	2.177	2.236
24	Baked at 400° C. for 18 hours	+0.277	....	.....	....
25	In cold water 7 hours and boiled 18 hours	+0.363	.....	2.182	2.245
26	Frozen while wet, at +29° F. and measured while frozen	+0.381	.....	.....	.....
27	After thawing out under water	+0.368	.....	.....	2.226
28	Boiled for 48 hours	+0.390	.....	.....	2.238
29	Heated rather rapidly to 500° C. and held there 18 hours	+0.529	.....	.....	....
30	In cold water in vacuum ½ hour	+0.725	.....	2.145	.....
31	Boiled 2 periods of 24 hours each	+0.752	.....	.....	2.199
32	Baked at 550° C. for 18 hours	+0.482	....	.....	....
33	In cold water in vacuum ½ hour, then boiled for 2 periods of 36 hours each	+0.655	....	2.155	2.198
34	Baked to 600° C. for 18 hours	+0.482	.....	.....	.....
35	In cold water in vacuum ½ hour, then boiled 36 hours	+0.625	.....	2.151	2.195

The values obtained at low temperature agree very well with the commonly accepted value of 0.0000055 per degree Fahrenheit. Apparently this coefficient increases slightly up to 575°F., but becomes smaller at higher temperatures and at about 1500°F. becomes zero and shows a slightly negative value above that temperature. Further, the blocks heated to 1500°F. did not return to their original dimensions on cooling but retained an elongation of about 75 per cent of their maximum elongation. On heating these blocks a second time no sensible permanent elongation was evident.

The explanation of these results of Norton must be sought in the differing behavior of the colloid of the hydrated cement and of the sand and gravel mixed with it. A parallel study of the effect of heat on bars of neat cement and of 1:3 sand mortar by White([29]) gives the necessary explanation. The bar of neat cement used had been kept five years in air and water alternately, and had during that period behaved entirely normally. It was then subjected to heat at progressively increasing temperatures, the change in length after cooling was measured and the bar was then immersed in cold water for a long period or in hot water for a shorter period to observe how fully it recovered its original length. The changes are fully explained in Figure 6 and the accompanying key. The bar became shorter in every case after each heating and cooling operation. It recovered part of the shrinkage on immersion in water but kept successively growing shorter with each cycle until it had shrunk 4.61 per cent. The dehydration of the colloid by heat had caused this bar of neat cement to contract just as a clay brick contracts in burning.

The behavior of the bar of mortar made from 1 cement and 3 sand appears in marked contrast to that from neat cement. This bar was cast integrally with a bar of neat cement and formed one of the 146 series illustrated in Figure 3. The baking tests started after the bar had been kept three and one-half years alternately in air and water at room temperature. The changes are shown in Figure 7 and the accompanying key. It will be observed that the neat portion of the bar was restrained from continued contraction by the sand portion whose length became progressively longer. The neat portion cracked

after a few alternations and was sawed off. The sand portion contracted somewhat after every heating with the exception of one test where the heat was applied too rapidly, but it always remained longer than its original measurement, so that it became progressively longer with each cycle.

These results agree entirely with those of Norton although they were carried out under entirely different circumstances, and they are sufficient to explain the changes in the thermal coefficient observed by him. When concrete is heated to high temperatures the gravel and sand expand but the colloid shrinks. These opposing tendencies cause the rate of expansion of the concrete to become less and ultimately to become zero with increasing temperature. The richer the concrete is in cement the less should be its coefficient at high temperature. When the heated concrete cools, the particles of sand and gravel lock in their expanded positions, so that the bar does not return to its initial length on cooling. A second heating will not cause it to vary much for there will be no further changes in the colloid. If, however, the colloid is allowed again to hydrate so that the interstitial spaces become again filled with dense colloid there will be expansion due to the development of the colloid and further expansion on heating, so that there will be a progressive elongation of the whole mass with each succeeding cycle.

In the tests both of Norton and of the author, the pieces heated were allowed to remain in the furnace until they had come to equilibrium with their surroundings. In the case of a conflagration where a very intense heat exists for a short time, the outer coat of the concrete may flake off from the core which has not been heated, on account of the differential expansion. This dehydrated colloid of the cement is, however, capable of reacting again with water and may with advantage be thoroughly soaked after it has cooled. It will then regain a considerable measure of the strength which is lost in the heating. This, of course, only holds where the mineral aggregate has been unaffected by the fire and does not hold good where limestone may have been heated until it has been converted into lime.

EFFECT OF FREEZING.

Very few actual measurements of change of volume during the freezing process have been reported. Rudeloff and Sieglerschmidt([21]) examined the behavior of blocks made from a mixture of 1 cement to 3 sand mixed to the consistency of damp earth, removed from the moulds after twenty-four hours, put into water and placed in the cold chamber whose lowest temperature was only—9.4°C. (=+15.1°F.). Several days were taken in cooling the blocks to this temperature and in again thawing them out so that changes due to freezing are difficult to dissociate from those due to the progressive hydration of the green cement. Rudeloff and Sieglerschmidt, in their summary, disregard the slight exceptions noted and state that the usual coefficient of thermal expansion also holds for temperatures below the freezing point.

White([29]) has reported the results of four successive freezings of bars of neat cement and 1:3 sand mortar, which had been kept alternately in air and water for from three to six years. All of the bars had been in water two months prior to the tests so that the colloid was fully developed. None of the neat cement bars showed any expansion on freezing. On the contrary they contracted at a rate which was nearly identical with that of cast iron. After four successive freezings and thawings under water the bars were entirely unaffected in length and specific gravity. The points where these tests were made are indicated in Figure 2 by asterisks. The bars of 1:3 sand mortar did not agree so well on the freezing tests among themselves. Two of the three 146 C1 and C2 of Figure 3 agreed with the neat bars in showing practically the same contraction as cast iron. The third bar 150 G of Figure 3 showed not only a relative but an absolute expansion when frozen, which was retained after thawing. This bar, after being frozen and thawed five times, showed a permanent elongation of 0.130 per cent, as is shown in Figure 3 by the asterisk. Its behavior cannot be laid to the cement, for one of the neat bars, which behaved normally on freezing (150 B of Figure 2), was made from the same cement. An inspection of the life history of these three bars, as shown in Figure 3, shows that 150 G had

been kept dry for a great share of its life and that although fully saturated with water at the time it was frozen, it was still 0.08 per cent shorter than its initial length. A comparison of changes in weight and specific gravity of these bars makes it evident that the pores of this bar were not completely filled by colloid but contained liquid water which, by its expansion, caused permanent expansion of the bar. It is apparent that the action of frost on concrete will depend on the amount of liquid water which it contains. This, in turn, is a function not only of the original composition of the concrete but also of the extent to which its colloid is developed at the particular time of freezing. Further experiments are under way to confirm this point.

EFFECT OF VOLUME CHANGES IN CONCRETE PROTECTED FROM THE WEATHER.

The coefficient of thermal expansion has been well recognized by engineers and it has been a matter of surprise that long concrete buildings have not shown more elongation in summer than is evident. This lack of visible effect is largely due to the parting joints left intentionally, or the cracks which form spontaneously as the concrete dries, as has been discussed in an earlier section. The writer is familiar with a building in whose attic are two continuous beams, each extending over 100 feet without connection with any other beam and without being over any columns. These beams are cracked at fairly regular intervals of fifteen feet throughout their length. It was shown in an earlier section that a shrinkage of 0.04 per cent might be expected in a 1:4 concrete hardening in the air. This is as much as would be expected from a temperature variation of 75°F. A rise in temperature of 75°F. above the mean temperature at which the concrete hardened, would, therefore, do no more than close these cracks.

This paper can not take into account the strains set up in reinforced concrete through the shrinkage of the concrete. The experimental work on this subject is extraordinarily meager. The subject has been discussed by Emerson(11), Considère(30), Rabut(27) and most thoroughly by Shitkewitsch(24).

EFFECT OF VOLUME CHANGES IN CONCRETE EXPOSED TO THE WEATHER.

It is evident from what precedes that concrete expands when heated and contracts when cooled and that its change of volume when frozen may be either an expansion or a contraction. It expands when wet and contracts when dry, the magnitude of the change varying with the proportions of the cement in the concrete and its previous history. The richer the concrete is in cement the greater will be its change of volume under the influence of moisture. Concrete which has been exposed to water for a long time will have its colloid fully developed and will resist freezing, but it will shrink more when dried than the porous concrete with its colloid not so well developed. The volume changes due to fluctuation in moisture are in general greater than those due to change from winter to summer temperature. It follows that concrete placed where it is continually dry or continually wet will not be subject to nearly as great alternations of volume as concrete which is exposed to the weather.

Concrete sidewalks and pavements are subject to conditions whose severity depends on the climate, rainfall and location in which they are placed. If the climate is generally dry, partings left for that purpose between the slabs will open or cracks will form of their own accord on account of the contraction of the concrete as it dries. Since the contraction on drying is greater than the expansion due to ordinary summer heat, the slabs of pavements laid in dry climates will rarely attain the length they had when initially cast, and, therefore, provision for expansion is unnecessary. The case is different with concrete exposed to the weather in climates with sufficient rainfall to keep the ground on which the concrete rests wet for periods of several weeks at certain seasons of the year. Ordinary concrete is porous enough to absorb rain and to suck water from damp ground readily. The Bureau of Standards ([28]) has made some measurements of concrete highway slabs which indicate the effect of the combined actions of temperature and moisture. These measurements were all started within a few days after the concrete was placed, and some of the initial

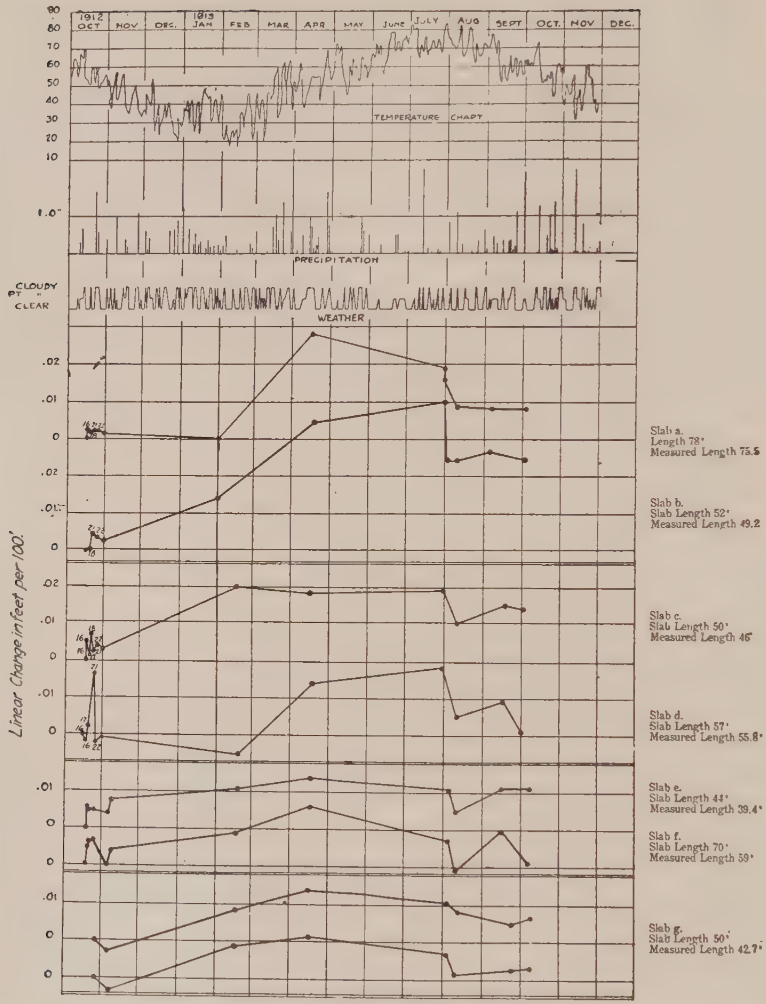

Fig. 8. **Measurements of Changes in Length of Slabs of Concrete Highways made by the Bureau of Standards.**
(From Proc. Nat. Conference on Road Building, Chicago, 1914.)

fluctuations may be due to changes attending the early hardening processes of the cement. The changes in length of these slabs are shown graphically in Figure 8, the plate being taken from the report of the Committee on Contraction and Expansion of Concrete Roads to the National Conference on Concrete

Road Building at Chicago in 1914. It might be expected that concrete laid in October, as these slabs were, would suffer marked shrinkage during the fall and winter months, but seven of the eight slabs were longer when measured in midwinter than in October, attained their greatest length in the spring and showed marked shrinkage during the hot summer month of August. One slab, not included in Figure 8, showed continuous contraction. The explanation given is that this slab was constructed on an old concrete road, in which the cracks were sealed with cement mortar. A half-inch of clay was laid upon this old road to prevent the new concrete from bonding to it, and the new road was laid upon this foundation. These conditions prevented the absorption of water from below but allowed evaporation from the top, thus causing a shortening of the slab. With this one exception all the slabs showed a general expansion instead of a contraction, as would have been the case had they been laid in a rainless region. The maximum expansion shown in the one year is 0.04 per cent and it is evident that to provide for this, expansion joints should be provided. If this is to be the total expansion of these roads the expansion joints need not be large.

It is, however, a matter of common knowledge that cement sidewalks after several years show marked expansion with results similar to those shown in Figure 9, reproduced from Boynton's Portland Cement Sidewalk Construction. This progressive expansion has been explained in a preceding section as due to a locking of the particles of aggregate in the expanded position, which allows water to enter and hydrate portions of hitherto untouched cement. The only apparent way to prevent this progressive expansion is to provide liberal and frequent expansion joints or prevent the concrete from becoming water soaked. If each block is made a separate unit there will sometimes be differential expansion of the base and top resulting in a dished slab. After frequent alternations the top may split off the base, either the top or base or both becoming shattered in the process. These phenomena are to be observed in sidewalks but have not as yet been noted much in concrete highways, for few of them in this country are over five years old. However, it is reported that on Long Island Motor Speedway([33]) some

expansion joints have had to be inserted where expansion had damaged the pavement, and that there are many transverse cracks.

The concrete pavements in Wayne County in the neighbor-

Fig. 9. Expansion of Concrete Sidewalks.
(From Boynton, Portland Cement Sidewalk Construction.)

hood of Detroit, Michigan, were carefully examined by Rogers and Cox([25]) in the fall of 1913, and again a year later. A summary of their figures for 1914 is given in Table V. Their itemized report lists separately longitudinal cracks, transverse cracks, diagonal cracks and holes, but in this summary only the totals are given.

Table V. Results of Inspection of Wayne County Concrete Roads in 1914.

Name of Road	Number of 25 ft. sections inspected	Mixture	Year built	Percentage defective slabs, 1914	Percentage increase in defective slabs in the last year
Woodward	209	1-2½-5 1-2-3	1909	72.7	11.0
Grand River	61	1-2½-5 1-2-3	1909	42.6	19.5
Woodward	252	1-2½-5 1-2-3	1910	36.9	13.5
Grand River	341	1-2½-5 1-2-3	1910	49.0	19.1
Michigan	481	1-2-4	1910	59.9	7.5
River	149	1-2-4	1910	48.3	10.0
Gratiot	326	1-1½-3	1911	21.8	13.2
Grand River	575	1-1½-3	1911	21.5	12.8
Michigan	1570	1-1½-3	1911	26.4	6.6
River	434	1-1½-3	1911	51.6	9.2
Grand River	1208	1-1½-3	1912	14.9	4.5
River	213	1-1½-3	1912	24.4	14.6
River	208	1-1½-3	1912	21.2	9.2
Fort St. Road	450	1-1½-3	1912	14.7	8.7

While undoubtedly there are many causes contributing to the defects enumerated in this report on the Wayne County Concrete Roads the very high proportion of defective slabs in roads that have had only from two to five years' service indicates that forces are operative which are not well understood, or provision would have been made to prevent the defects. The progressive expansion and contraction due to moisture, the loss in strength due to moisture changes and the harmful effect of

water freezing in cracks started from one of the other causes are undoubtedly influential factors. It cannot be doubted that these same factors are influential in causing the disintegration of sea walls exposed to the action of the tides. The case is complicated by the possibility of chemical action of the sea water, but the fact that the walls continually immersed and the walls never immersed are only slightly disintegrated while the intervening wall, which is alternately wetted and dried, may be completely decomposed, is clear evidence of the part which alternate wetting and drying plays.

THE EFFECT OF VOLUME CHANGES IN CONCRETE.

The most elaborate study of the mechanical effects of these volume changes and the methods of obviating them has been made by Professor Shitkewitsch(24) of the Royal Engineering Academy in St. Petersburg. He summarizes his paper as follows:

1. The smallest volume changes are found in lean, porous concretes made from first class aged cements and stony aggregates such as brick, limestone and granite.

2. To obtain the greatest uniformity in massive concrete and to prevent horizontal cracks, there must be an uninterrupted deposition of the concrete and an interlocking of the surfaces of the layers as deposited.

3. The limiting dimensions of concrete blocks vary in every case with the properties of the concrete and the influence of moisture, temperature and structural conditions. The greatest dimensions of blocks can be obtained by minimizing the structural conditions which hamper a free change in volume of the concrete.

4. It is necessary in concrete structures to provide the greatest possible independence of the single parts which are subjected to varying load and fluctuating atmospheric conditions; which can be attained by cutting the whole structure into single monoliths separated by horizontal and vertical partings.

5. For the preservation of detached concrete structures from the destructive influence of atmospheric agencies, it is necessary to have

(a) a heavy facing on external wall surfaces and

(b) a water proof cover for all other open surfaces.

The volume changes in concrete are far more serious in their consequences than is usually believed. Concrete continually wet and concrete continually dry or exposed only to changes in the humidity of the air and not exposed to too rapid

alternations in temperature will probably be so durable as to be worthy of the name permanent.

Unprotected concrete exposed to the weather in localities where it is liable to become frequently soaked with water and where it may also be frozen while wet can hardly be considered a permanent structure.

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DISCUSSION

Prof. A. H. Fuller,*** M. Am. Soc. C. E., said that in determining stresses from deformations in concrete, the time element must be considered. It was discovered that the deformations in a certain building were greater than the deformations in laboratory tests for the same loading. When these laboratory tests were repeated, with loads applied gradually, about as in practice, the deformations were greater than showed by ordinary experiment. These loads were continued for five months and then the deformations were three or four times those obtained when the maximum load was first reached. Prof. Fuller.

Mr. F. G. Baum*, M. Am. Soc. C. E., stated that he is familiar with the work of two arched dams, one 225 ft. high and the other 166 ft. high. Both dams were completed in about two years, and the deformations were less than expected. Measurements were taken through several years. In the first dam, contraction joints were placed every fifty feet, the dam being 500 ft. long. In the second dam, only two joints near the ends were used. The end sections were poured last, in cold weather. In each case a slight crack appeared, which was only 1-16 of an inch at the maximum. Mr. Baum.

The time of mixing concrete affects the strength as well as the elasticity. Raising the time of mixing one or two minutes may increase the strength of the product by 50 percent.

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Paper No. 104

WATERPROOF CONCRETE.

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I. CONCLUSIONS.

The extensive use of concrete in building construction has developed many problems and not the least of them is "waterproof concrete", which has occasioned much discussion as to the merits of the various waterproofing materials and methods of waterproofing. The problem is to protect a structure of concrete against the penetration of moisture into or through the mass, thereby preventing dampness (affecting the health and comfort of those who occupy or use it), the loss of the material stored in or conveyed by it, and destructive action by frost, alkali or electrolysis.

There is also the problem of the insulation of walls to prevent dampness and discoloration resulting from the condensation of moisture on their surfaces.

The following conclusions are based on the experience of the writer in this country and Europe, comprising a study of laboratory investigations and tests, and of the practice in the erection of various types of reinforced concrete structures; on correspondence with the leading engineers in this country; and on a review of the current English literature on the subject. In addition to this, the very extensive series of tests made under the writer's direction while in charge of the structural materials investigations for the United States Government have afforded him information as to the comparative value of the

various materials and methods of waterproofing, which is set forth in this review of the present current practice in the erection of structures of waterproof concrete.

The writer believes that the facts presented justify the following conclusions:

1. Failures to secure waterproof concrete result from the formation of cracks due to defective design and faulty details, and to poor materials and workmanship. It may be generally stated that workmanship and design, inadequate to secure water-tightness in addition to strength, are responsible for practically all the failures.

2. Concrete properly proportioned, mixed and placed, with adequate provision for shrinkage and contraction cracks, and for construction joints, will be waterproof. Unless properly reinforced, cracks may occur even though the concrete itself is water-tight.

3. So many successful structures have been built which meet all conditions of service, that there is no question that it is possible to build waterproof structures of concrete without the use of waterproofing coatings, membranes or integral compounds.

4. Tests show that the waterproofing materials which are added to concrete do not increase the density of the cement paste which fills the interstices between the aggregates.

5. Impermeability of concrete can be better secured through the addition of cement, than by any foreign material.

6. The aggregates for concrete, that are readily available for use on engineering work, possess sufficient uniformity to permit the proportioning and grading necessary to secure waterproof concrete at no increased cost.

7. The same care and attention to details in the design and erection of waterproof concrete structures that are given in the application of the methods of waterproofing now in vogue, would greatly increase the number of successful examples.

8. There is no waterproofing material that is entirely satisfactory or permanent—in time they all must be renewed; a

structure of waterproof concrete, properly designed and built, does not lose its water-tightness, and its life is practically unlimited.

9. Very few waterproofing materials or methods of waterproofing, at present in use, possess appreciable merit, and their use is generally a waste of money.

10. To secure a water-tight structure of concrete, either without or with special waterproofing, it is essential that the details shall be carefully planned. Materials must be carefully placed by skilled workmen and proper provision must be made to care for construction joints, and for shrinkage and temperature cracks, through the use of an adequate amount of properly placed reinforcement.

11. In practice where the structure is of such magnitude as to make it inconvenient to cast it as a monolith, it is frequently preferable to provide some form of impervious material to seal it, so as to prevent leakage at the cracks or construction joints.

12. Where concrete is placed against metallic plates, as for example in bridges, it is impractical to prevent leakage between the steel and the concrete, and resort must be had to some impervious elastic material to carry water away from the resulting seam to the drainage channels.

13. The surface of concrete may be water-tight, and yet absorb water, which may result in objectionable surface discoloration and efflorescence. Surface porosity should not be confused with permeability, although, if the pores are continuous, water may eventually penetrate through the concrete by capillary action. The treatment of the surface of concrete with a waterproof coating to render it capillary positive may be desirable under certain conditions.

14. Where such material is used, it should be tested to determine whether it is free from elements that may seriously retard the hardening, cause disintegration, or in other ways have a deleterious action on the concrete.

15. It is for the engineer to determine whether it is more advantageous or economical to use some special method of waterproofing, or to direct his energies to secure concrete pro-

portioned, mixed and placed in such a manner as to obtain the water-tightness he desires.

16. The time is rapidly approaching when the engineer will use the same care in selecting, grading, proportioning and mixing the aggregates of concrete, that he now uses in determining the quality of the cement, or in the application of various methods of waterproofing; and under such conditions, with proper inspection and care in erection, water-tight structures will be readily built without special methods of waterproofing.

17. The writer believes that the time will come when failure to secure a water-tight structure will be a reflection on the ability of the engineer under whose direction the work is executed.

18. Where waterproofing material is used, workmanship is more important than the material. The waterproofing material must be thoroughly and uniformly distributed throughout the mass of concrete, and this can only be accomplished by mixing for a considerable length of time. The same amount of care taken to secure properly proportioned, graded, mixed and placed concrete would result in the same degree of waterproofness without the addition of waterproofing materials.

19. Bituminous material grows brittle and hard with age, loses its elasticity, cracks and thus becomes of little value as an impervious seal.

20. Joints and cracks are the usual source of leakage. These construction seams receive little consideration at the time of the design or during the erection of the structure. If adequate precaution was taken to secure water-tight joints, the structure would be water-tight under practically all conditions of service, since experiments show that 2 in. of properly proportioned, mixed and placed concrete is water-tight under considerable pressure.

21. The development of a successful joint—one that is cheap, practical and simple in installation—offers a most important field in the development of water-tight concrete structures.

22. Placing a layer of impervious material in the concrete will not make the concrete, *per se,* waterproof.

23. Concrete of maximum density must have adequate reinforcement to care for the shrinkage and temperature stresses, in order that the structure may be water-tight.

24. Waterproof concrete, properly hardened, is unaffected by the action of frost, sea water, alkalies, or electrolysis.

II. GENERAL DISCUSSION OF THE SUBJECT.

1. Necessity for Waterproof Concrete.

The structures of concrete built a number of years ago, which often form the basis of an opinion on the question of water-tightness, were usually of dry concrete which did not contain sufficient water to permit working to the proper density. No amount of ramming of dry concrete will eliminate voids, nor increase the density. On the other hand, experience has shown that a wet concrete that is stirred or agitated, or better still, vibrated, is much more dense by reason of the consequent elimination of the air and water voids. If the surface of such concrete is given a granolithic finish it is practically impervious to water.

Small masses of concrete can be easily made impervious, but to make a large structure water-tight, at reasonable cost, is a problem not of the imperviousness of the material, but the water-tightness of the entire structure. Placing a layer of impervious material in concrete does not make the concrete impervious. It should be borne in mind, however, that while the concrete may be impervious, a crack may develop which will cause leakage.

''Waterproof'' is a relative term,—no material is strictly ''waterproof''. All masonry materials are more or less porous and permit the absorption and penetration of water. Moisture will be absorbed or pass through concrete if the pressure be sufficiently great or prolonged; continuity of pores or voids, thinness of the mass, all have a bearing on water-tightness. To develop capillary action, it is necessary that the pores be very minute and continuous; and under such conditions, water in time may entirely permeate the mass. As these pores increase in size, the capillarity decreases until it becomes zero. The void spaces in concrete, therefore, do not produce capillary action,

but do result in permeability. A cement surface may absorb water, although the mass may not readily be permeated. Under these conditions, the use of some water-repellent material may be advantageous to retard or prevent the penetration of water into a dry mortar or concrete.

The design of a water-tight structure of reinforced concrete requires considerable judgment and skill and a careful study of the conditions governing the use to which it is to be put. It frequently happens that the subject of waterproofing is exploited by people lacking an adequate conception of the purposes for which the structure is intended, and who, therefore, advocate the use of materials and methods of waterproofing not only wholly unsuited to the conditions, but frequently detrimental to the concrete.

Impermeability of concrete is a question of density and is secured by filling the pores or voids in the concrete with finely divided, insoluble material. This can be best accomplished by properly grading the aggregates, and it may also be obtained by the addition of certain materials whose chief function is to fill the voids. A structure of concrete to be absolutely water-tight must be solid, and its imperviousness decreases as its solidity decreases, or as the void spaces increase. The permeability of concrete is also affected by the permeability of the aggregates of which it is composed.

While it is true that the more porous a mortar, the less impermeable it is, this principle cannot be applied to concrete, because the term as generally accepted refers to void spaces so large that the capillary action is insignificant. Concrete exposed to the free passage of water gradually becomes tight; the decreased permeability is due to the filling of the voids.

The writer does not wish to be understood as endorsing the use of the many waterproofing compounds which are now on the market. On the contrary, he believes it is quite practicable to make concrete water-tight, under the usual conditions of service, by properly proportioning, mixing and placing it without resort to waterproofing material of any character. It was for the purpose of establishing this fact that he inaugurated, in the Structural Materials Testing Laboratories of the United

States Geological Survey, the series of tests of some 40 odd waterproofing compounds, the results of which are published in Technologic Bulletin No. 3 of the U. S. Bureau of Standards. The results of these tests show that only a very small percentage of these compounds possess appreciable merit. The writer further believes that good engineering practice does not require the use of such material, and that concrete can, without increased cost, be rendered sufficiently water-tight for all practical purposes.

It should be noted that waterproofing materials, especially coatings, which prevent moisture from penetrating the mass, or the excess water used in mixing from drying out, may prevent the proper hardening of the mortar or concrete.

Where waterproofing materials are mixed with the concrete, they should be of such a character that they will have no deleterious effect upon the mortar or concrete, nor seriously affect its hardening. They should contain no organic matter, and, in any event, the quantity should be limited in percent. Where unknown materials are used, tests should be made to determine,

1. Their effect upon the strength of the mortar or concrete.
2. Their behavior under wide ranges of temperature.
3. The effect of acids and alkalies on them, and
4. Their effect on the life and bond of embedded metallic reinforcement.

In designing water-tight structures it is necessary to carefully consider the conditions under which the structure is to be used. It is evident that a reservoir or tank with only a few feet of water need not have the same resistance as one under a head of 40 or 50 feet; and it makes considerable difference whether a conduit or pressure pipe is under a 10- or 100-foot head. Concrete structures are designed to meet conditions in which water would be objectionable because of damage to materials stored in them, or where dampness in the walls would be objectionable to the occupants or would result in objectionable surface discoloration, and also to resist disintegration resulting from frost, sea water and alkali action and electrolysis.

The structures usually waterproofed are floors of bridges and arches over city streets, to prevent leakage of ballast water; roofs and walls of subways, to prevent leakage of ground water; basement and building walls, to prevent leakage of rain or ground water; reservoir and tank walls, to prevent loss by leakage. Some conditions may permit a slight leakage (a reservoir may leak a few gallons daily), other conditions will not permit leakage (a bridge floor must not allow water to drip), and there are circumstances where moisture causing a stain or discoloration would be objectionable. In case of fire, and in some processes of manufacture, it is necessary to have water-tight floors, so as to confine water to the floor in which it is used and prevent damage to the contents on the floors below.

2. Method of Attacking Problem.

Concrete must be so designed as to provide for cracking, due to unequal settlement, and for shrinkage and temperature changes. Since a structure of concrete may be so designed as to successfully meet all requirements as to construction, service, and cost, the next matters to be considered are the methods which may be used to secure water-tightness.

The various expedients for making concrete waterproof are:

1. Proportioning, mixing and placing concrete to secure a maximum density.
2. Plastering the exposed surface with a rich mortar (1 to 1, or 1 to 1½).
3. Incorporating waterproofing material in the concrete.
4. An enclosed impervious seal to keep the water from the concrete.

Perhaps one of the most important points in design, is the use of reinforcement to so distribute the stresses occasioned by undue settlement, shrinkage and temperature as to render the resulting cracks sufficiently small and non-continuous to prevent the passage of water through the concrete. The structure must also be so erected as to avoid construction joints and seams which might prove a source of leakage. The conditions of use may be such as to require unusual precautions to

secure water-tightness; on the other hand, it may only be requisite to prevent the penetration of water into the mass so as to avoid dampness, efflorescence or undesirable discoloration. There are conditions under which it may be necessary to reduce the surface capillarity by rendering it water repellent, thus preventing moisture from traveling over the surface, bringing out soluble salts and producing discoloration.

In some of the arid portions of this country where the soil contains large quantities of certain alkalies, it is especially necessary that the surface of the concrete shall have a low absorption; since the rapid formation of crystals in the pores, due to the extreme rapidity of evaporation, sometimes results in the breaking of the bond in the concrete and its subsequent disintegration.

Under such conditions, the most critical part of the structure is at and just above the ground line, or that zone in which the alkali salts crystallize in the pores of the structure under the conditions of rapid evaporation. It should be noted, however, that this destructive action affects brick, stone and other building materials, in proportion to their porosity.

To satisfactorily resist the action of alkalies, it is necessary that the exterior surface of the concrete shall be impenetrable, and this may be accomplished by a granolithic finish or by surface treatment.

In providing against dampness, the conditions for walls below ground are different from those for exterior and interior walls above ground. The dampness, so frequently referred to by writers on waterproofing, that is found on concrete walls results from the condensation which is to be found on all walls of masonry that are not properly insulated. This is due generally to defective ventilation and is the natural result of warm moist air coming in contact with the colder wall surface.

The foundation walls of a building where considerable water is present, unless of dense concrete, are likely to become saturated with water, due to capillarity of the more or less porous mass. This, however, is a condition which ought not to occur, since the concrete can be made impervious.

If a basement of a building is under a head of water, it may be necessary to make the exterior foundation walls water-tight to prevent the penetration of water into the building; this may be accomplished through the use of properly proportioned, mixed, placed and reinforced concrete; the concrete should be placed continuously, so as to secure a monolithic structure. It frequently happens in buildings of large size that the walls must be built in sections, and it is not possible to secure a monolith. It may, therefore, be more convenient to use an impervious material to completely seal the structure against the passage of water. This expedient is frequently resorted to not because a water-tight concrete structure could not be otherwise secured, but because under conditions of construction, by reason of inability to eliminate the construction joints, it is considered more advantageous to resort to this method. However, it should be noted that such seals themselves must be impervious and unbroken, as punctures or openings will permit leakage.

3. Practicability of Securing Waterproof Concrete.

Ample proof is afforded of the practicability of making concrete water-tight, at a reasonable cost, by the numerous examples of structures already built, of which a number are given in this paper. It is not difficult nor is it impracticable to so proportion, mix and place concrete as to reduce the voids to a minimum and thus secure an impermeable mass. It is necessary to so design and care for the stresses due to temperature and shrinkage as to prevent the occurrence of cracks, and to so erect the structure as to avoid construction joints which may be a source of leakage.

III. MEANS OF SECURING WATERPROOF CONCRETE.

1. Impervious Concrete.

Among the methods available for securing water-tight structures, the most rational is the use of impervious concrete, properly proportioned, mixed and placed, and adequately reinforced to prevent cracks. For moderate pressures, concrete can readily be made impervious when mixed in the proportion of one part of Portland cement to six parts of aggregates; for

higher pressures, a somewhat richer mixture should be used, although not richer than 1 to 4.

In order to secure an impervious concrete, the following precautions should be observed:

1. The concrete should be composed of dense, impervious aggregates, carefully graded from fine to coarse, and so proportioned as to secure a concrete of maximum density.

2. There should be an excess of cement.

3. It should be thoroughly mixed with enough water to produce, after a prolonged period of mixing, a viscous consistency, that will admit of being conveyed without a separation of the aggregates, and which can be readily tamped without bringing an excess of water to the surface.

Consistency and mixing are the most important factors in securing waterproof concrete. In former days it was the practice to mix with so little water that dry and extremely poor concretes were the result; it is the present practice to mix concrete entirely too wet, and water is impounded, which is drawn to the surface by evaporation, leaving void spaces in the concrete which become channels for the penetration of water. The prolonged mixing of a medium wet concrete is the most satisfactory means of eliminating air and water voids, and developing colloids, which tend to increase the cohesion and density. The German practice of working comparatively dry mixtures for a long time, or until they are brought to the above described consistency, in the writer's opinion explains the dense impervious concrete which is obtained, especially in the manufacture of cement products. A concrete thus mixed, with an excess of cement, and placed continuously so as to secure a monolithic structure, will be water-tight.

Numerous tests have been made to determine the permeability of cement mortars and concretes, especially those that were made in the Structural Materials Testing Laboratories of the United States Geological Survey, which have developed the following facts:

1. Concrete having the greatest density is the least permeable.

2. The impermeability increases with the quantity of cement.

3. In mortars and concretes of the maximum density, the flow decreases with time.

4. Fine-sand mortars are less permeable than those of coarse sand.

5. Permeability is largely effected by the relation of the fine to the coarse aggregates.

6. Richness of the mortar has less effect in decreasing the permeability in mortars of dry consistency than in mortars of quaking consistency.

7. Absorption is less in mortars of quaking consistency than in those of dry consistency.

8. The permeability decreases with the thickness of the mortar or concrete.

9. The permeability for similar mortars is the same for thicknesses of 1, 2 or 3 inches.

10. Three-inch test pieces of concrete are slightly less permeable than two-inch test pieces.

11. Permeability increases as the pressure increases.

12. Permeability decreases with age and quantity of water passing through the mortar or concrete.

While fine-sand mortars are less permeable than coarse-sand mortars, they are acted upon by sea water to a greater extent.

Trowelling the face of concrete exposed to the action of water, to a granolithic surface, is a very effective means of waterproofing.

Mortars and concretes which permit a slight seepage of water will generally become water-tight in time, due to the filling of the pores with the fine material carried by the water.

One of the most effective ways of making mortars and concretes impervious, is through the use of an additional quantity of cement.

2. Use of Foreign Material In or On the Concrete.

(a) Integral Compounds (Powders). The addition of finely divided material such as hydrated lime, pozzolana, clay, sand, etc. (known as inert fillers), has for its object the filling of the pores in the mortar or concrete; such materials have a tendency

to increase the absorption of water. Materials containing stearic acid or other greasy substances are known as water repellents, since they have a tendency to repel water, or prevent its absorption. So long as these materials do not affect the hardening of the cement, it is immaterial whether the action is physical or chemical.

The addition of pure clay, finely powdered, free from any vegetable matter, materially increases the water tightness of concrete by acting as a void filler. The effect of clay of the proper quality varies with the richness of the mortar or concrete; it has no value in well-proportioned concrete, since the extra cement supplies all the fine material necessary to fill the voids. Clay is difficult to use because of its hydroscopic qualities; colloidal clay is not readily obtained, must be dried and finely pulverized, and is not water repellent. In moderate amounts it has no effect on the strength of the mortar or concrete, but it tends to retard the hardening. While laboratory tests show that clay increases the strength of concrete, nevertheless, in actual construction the use of this material is bad practice, since it tends to ball, to coat the aggregates, and reduce the strength of the concrete.

Hydrated lime fills the minute pores in concrete, thereby producing a dense mass impervious to moisture—the action is largely mechanical.

The use of these inert fillers is not to be recommended, since the addition of an equal quantity of cement will produce more successful results, except, perhaps, in the case of the water repellent materials which may be used under certain conditions to reduce the surface absorption.

The experiments made by M. Feret, of France, showed that cement mortars kept in air were weakened by the introduction of pozzolana and that the action was in part chemical. The same strength was obtained, however, by the use of inert material of the same degree of fineness. Mr. Poulson, of Denmark, finds that the use of pozzolana increases the strength and density of the mortar or concrete. This mechanical addition may increase the strength as well as the permeability by increasing the density.

The experiments in the Government laboratories indicate that void fillers up to 20% generally increase the compressive strength of the mortar or concrete.

The practice of adding dry powder before mixing the concrete is open to the objection, that to be effective, it must be thoroughly disseminated in the concrete, and if this is not done, the mortar or concrete will vary considerably in density and strength. If the same amount of mixing was given the mortar or concrete without the addition of these materials, the density and strength would be more satisfactory.

(a) Integral Compounds (Liquids). Various liquids are added to mortars and concretes to render them waterproof; among these may be mentioned oils, lime and soap, and alum and soap solutions; the water-tightness in this case is due to the gelatinous coating about the particles.

Combinations of liquids and powders are also used, in which case the waterproofing is the result of precipitation of insoluble compounds in the voids. The soap and alum mixtures deposit a flocculent, insoluble compound in the voids, generally a gelatinous mass of aluminum silicates, thereby rendering it impervious.

The addition of mineral oil to concrete has not proved satisfactory, and the results thus far obtained show that it requires so much care that there is no advantage in using it, since the same amount of care taken in mixing mortars and concretes would make them impermeable without the use of waterproofing. The mineral oil is absorbed by the mortar or concrete and forms an emulsion with the cement; it is therefore water repellent, and decreases the surface absorption of water. In the alkali districts the treatment of the concrete surfaces with oil seems to indicate that they are less subject to the action of alkali than in the case of the untreated material.

(b) Coatings. It is the practice to render structures which absorb a considerable amount of water impervious through the application of surface coatings of various kinds, such as solutions of paraffin, soap and alum, bitumen, cement washes or coats of plaster of rich mortar. Such treatments are for the most part unsatisfactory, since the coatings are liable to come off or be destroyed. They may decrease surface por-

osity, but are inelastic and have no value, therefore, as a seal against the passage of water through cracks and construction joints.

The effect of coatings is to render the surface water repellent, thereby preventing dampness. Such treatment is also a preventive of discoloration and efflorescence. The paraffin solutions demand a volatile vehicle, which leaves the paraffin in the pores and as a coating on the surface; the coatings themselves are insoluble in acids and alkalies. The surface to which they are applied must be clean and dry, and the air must not be less than 50° F. In the case of paraffin coatings, the surface must also be free from sharp points and edges, and must be smooth, with all objectionable holes filled. Whatever material is used requires the application of several coats.

In connection with the New York subways, it was found by experiment that a suitable electrolyte (one or two percent solution of alum) used in the mixing water was advantageous in securing a water-tight concrete; replacing 5% of the sand with dry, finely pulverized colloidal clay was found to be equally advantageous. A combination of both methods was also found effective.

Many engineers state that a grout of neat cement will render concrete water-tight under small heads of water; in this case, the fine material is carried by capillarity into the mortar or concrete.

Surface coatings are inelastic and when cracking occurs they become inefficient. This is entirely apart from the durability of the material itself. Where joints or cracks exist, water-tightness is destroyed and it is difficult to bind the old and new coatings together in such a way as to restore water-tightness.

(c) Enclosed Impervious Seals. A method of securing water-tightness is through the use of enclosed elastic impervious seals, whereby water is prevented from passing into or through the concrete; when properly applied, they are effective remedies for leakage occasioned by cracks or construction joints.

These seals include all those methods of waterproofing by which masonry is covered with impervious material applied as

a mastic or in layers; these waterproofing layers, usually not less than five (the number depending on the pressure), are composed of felt, burlap, asbestos paper or similar soft, absorbent material which is not in itself waterproof, and which must be saturated with tar, asphalt, or some bituminous compound. The fabric is laid in overlapping layers with a coating of waterproofing material between them, and a coating on the outside of the completed seal.

In order to apply waterproofing of this character, the surface of the concrete must be clean, dry and reasonably smooth; it must be free from pockets or depressions in which water could collect, and proper drainage should be maintained. Such material should not be placed in cold weather, and where this is necessary, care should be taken to keep the bitumen hot. Swabbing should be done uniformly and at the proper temperature. Care should be taken to properly join the old with the new work; the former must be cleaned and must be thoroughly coated with fresh bitumen before the new is placed.

If the seal is punctured or becomes inelastic and cracks, leakage is almost certain to occur. Asphalt, tar and other bituminous materials are subject to alkali action, which destroys their life; many of these materials also become oxidized, and lose their elastic qualities. It is necessary, therefore, to protect them against the action of alkalies, water and air, and against any usage which will tend to puncture or otherwise destroy their continuity. They are generally laid on the concrete with a protection layer over them.

It is the opinion of many engineers that the use of impervious seals is desirable, since it is sometimes difficult to erect a water-tight concrete structure, and they can be applied without unusual precautions at a minimum cost. Since, however, these seals must of necessity be covered and protected, they are not readily accessible for repairs, and furthermore, it is difficult to trace the source of leakage, which makes such repairs generally expensive. Waterproofing of this character must be renewed after an interval of several years, since it loses its elasticity, and cracks, which reduces its efficiency.

IV. WORKMANSHIP AND INSPECTION.

Workmanship is a most important factor, either in securing waterproof concrete or in the application of waterproofing. Unskilful workmanship would render a very rich concrete pervious, while skilful workmanship would secure an impervious concrete with a very lean mixture.

In Germany, the writer has seen dry concrete, of a consistency that would be entirely impractical at the present time for use in this country, worked long and vigorously into a dense, water-tight mass.

The tendency here is to use entirely too much water in order to render the labor of mixing and placing easier, but, as a consequence, water as well as air voids are introduced into the concrete, which increase its porosity. The use of a less amount of water and much longer mixing, than is now the practice, would develop a gelatinous coating on the aggregates and increase the density, through the consequent elimination of the voids in the concrete. The most important step in the process of making concrete waterproof is the proportioning and mixing. Generally this matter has but casual inspection, and, as a result, the laborers under whose charge the work is performed, either from indifference or ignorance, fail to give it requisite attention, and the variable proportions and insufficient mixing result in poor concrete, lacking uniformity in composition and density.

When one considers the extreme care that marks the manufacture of steel or the fabrication of the structures in which it is used, one cannot but be impressed with the lack of similar care in the case of concrete, which performs similar functions as a structural material.

One of the most crying needs in concrete construction is the presence of an adequate number of competent inspectors throughout the entire erection. Such inspection is of the greatest importance in those structures which must be water-tight, and which of necessity require care and attention in proportioning, mixing and placing the concrete. Aside from the question of water-tightness, it is the writer's opinion that safety and good engineering require more inspection. The

presence of an adequate number of competent inspectors would result in more skilful mechanics, by reason of the educating influence of such supervision, which would eventuate in a much better class of structures.

The writer would not for a moment convey the impression that the present reinforced concrete structures in this country are in any way inferior to those erected in other parts of the world, but would point out the need for this inspection to prevent the occasional failures which result from the improper use of concrete, because of incompetency or lack of proper supervision. With adequate supervision a better grade of concrete would be secured, which would justify larger unit stresses than are now considered permissible, and there would be a consequent reduction in the size of the structures.

It is to be hoped that the developments of the future will result in more skilful mechanics and much more efficient inspection.

V. CRACKS AND CONSTRUCTION JOINTS.

The distribution of cracks resulting from shrinkage and temperature, so as to reduce their size and break their continuity through the structure, depends on the successful location of the reinforcement.

Small structures can be poured continuously, and will be sufficiently water-tight without the use of reinforcement. Where it is not possible to cast a structure as a monolith, construction joints become necessary, and leakage is almost sure to occur. The structure is generally built during the warmer months of the year, and when contraction occurs, the concrete shrinks at the joints; and as there is sufficient elasticity in the reinforcement to permit a very slight stretch, the bond in the concrete on either side of the joint breaks and makes an extremely fine seam through which water will pass.

The development of a proper method for making joints that will be water-tight is a problem worthy of the very best attention, and its successful solution will greatly decrease the difficulty in securing waterproof concrete.

Major John S. Sewell, of the United States Corps of Engineers, in discussing this question, states that: "In fortifica-

tion work the best practice to render the rooms damp-proof consists in finishing the extreme top surface of the concrete (over the passage in the fortifications and the rooms below), with a rich granolithic concrete laid in small independent squares, reinforced against cracking by small steel bars running both ways of the slabs. The joints between the slabs are caulked just like the seams of a wooden deck, and the whole exposed surface given sufficient slope to shed water quickly. Under these conditions, the water does not pass through the slabs themselves, and joints can be kept tight with little care''.

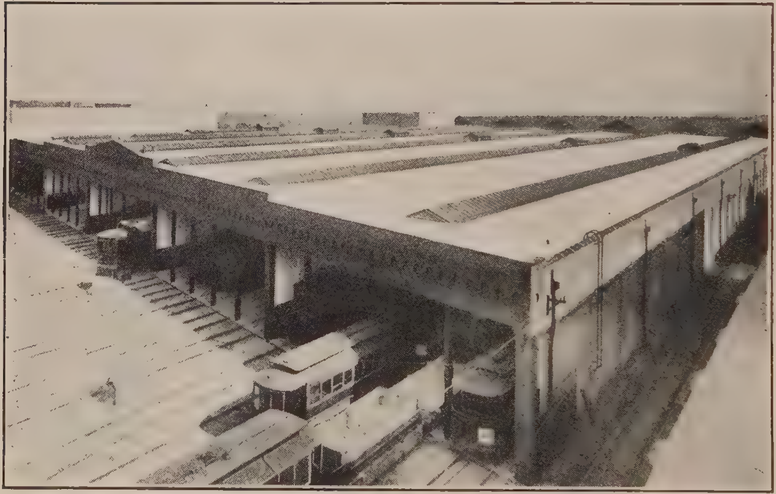

Fig. 1. Reinforced Concrete Roof of Car Barn in which no waterproofing was used; Chicago City Railway Co., 77th Street and Vincennes Road.

The principal difficulty in securing water-tight structures of concrete lies in making the construction joints water-tight. Too little attention is given to this matter, which is so important that it should receive as careful consideration as any other portion of the structure.

VI. EXAMPLES OF STRUCTURES OF WATERPROOF CONCRETE.

In the following pages will be found examples of various structures of concrete in which no waterproofing has been used, which have proved satisfactory, together with some structures in which leaks have developed at construction joints.

The process of molding cement products (drain tile, sewer pipe, building blocks, etc.) by machinery requires a dry mixture, to give the material sufficient stiffness to permit of its removal from the machine as soon as it is molded; the resulting product is extremely porous, has a high absorption, and is generally unsuited to the purpose for which it is intended. Waterproofing material, such as soap, hydrated lime, finely divided clay and sand, is used for the purpose of so reducing

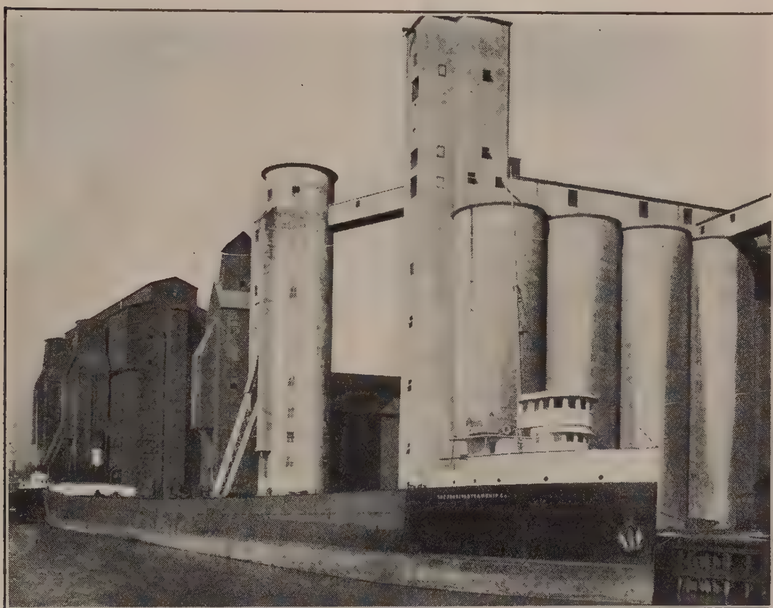

Fig. 2. Unwaterproofed Reinforced Concrete Grain Elevator. Frontier Elevator & Mill Co., Buffalo, N. Y.

the voids as to increase its water-tightness. The use of this material is not as efficacious as would be the use of an equal quantity of cement; these materials, while in a measure minimizing the objectionable weathering, are not all of a permanent nature, and in time must be renewed or otherwise the unsuitable conditions will return.

Cement products are being successfully made by the wet process, without waterproofing, which are in every way superior to those usually made by the dry process.

The interior walls of concrete structures in which grain or similar materials are stored must be dry and water-tight. In order to prevent the penetration of moisture, the structures are, wherever possible, built as a monolith, in order to avoid seams which would be a source of leakage. They must be designed with proper reinforcement for temperature and shrinkage, and the concrete must be of maximum density.

In the construction of roofs of reinforced concrete, it is often found desirable to use some form of impervious covering, usually of bituminous material, which will effectively prevent penetration of water into the structure below. The use of this material is, however, not necessary, and there are many examples of roofs of considerable extent which have been built without the use of waterproofing. These are generally finished with a richer mixture, so as to form a granolithic top surface. It is also a practice to build roofs of this character without reinforcement for temperature and shrinkage stresses, with the expectation of filling the cracks with bitumen as they develop. The illustration in Fig. 1 shows the roof of one of the car barns of the Chicago City Railway Company; it is 400 ft. wide and 500 ft. long, and is 5 in. thick; monolithic concrete was used throughout, without construction joints and without waterproofing. The writer understands that it is very satisfactory, and practically water-tight.

The problem of water-tight concrete becomes greater as the head of water stored and the extent of the structure increase. Where conditions permit, it is usual to build by continuous operation, and there are many successful examples of such construction. Nearly all the grain elevators which are built are not waterproofed, and where it was formerly the practice to build in sections, it is now considered better practice to build by a continuous pouring process to secure as nearly as possible a monolithic structure. In Fig. 2 is shown a Frontier elevator at Buffalo, N. Y., built about three years ago. No waterproofing was used in the construction, and it has proved entirely satisfactory. It has not even been washed down with a cement, which is frequently done not for the purpose of waterproofing, but in order to give the concrete a uniform color.

A similar elevator is the Harbor Commissioner's Terminal Elevator No. 3 at Montreal, Quebec, having a capacity of 2,600,000 bushels, and built entirely of reinforced concrete without waterproofing.

Fig. 3 Monolithic Concrete Storage Bin, Haubstadt, Ind. No waterproofing was used in this structure.

Concrete is now generally used in the construction of silos, and has proved to be very economical and suitable. A typical illustration of this use is given in Fig. 3—that of a monolithic reinforced concrete grain storage bin at Haubstadt, Ind.

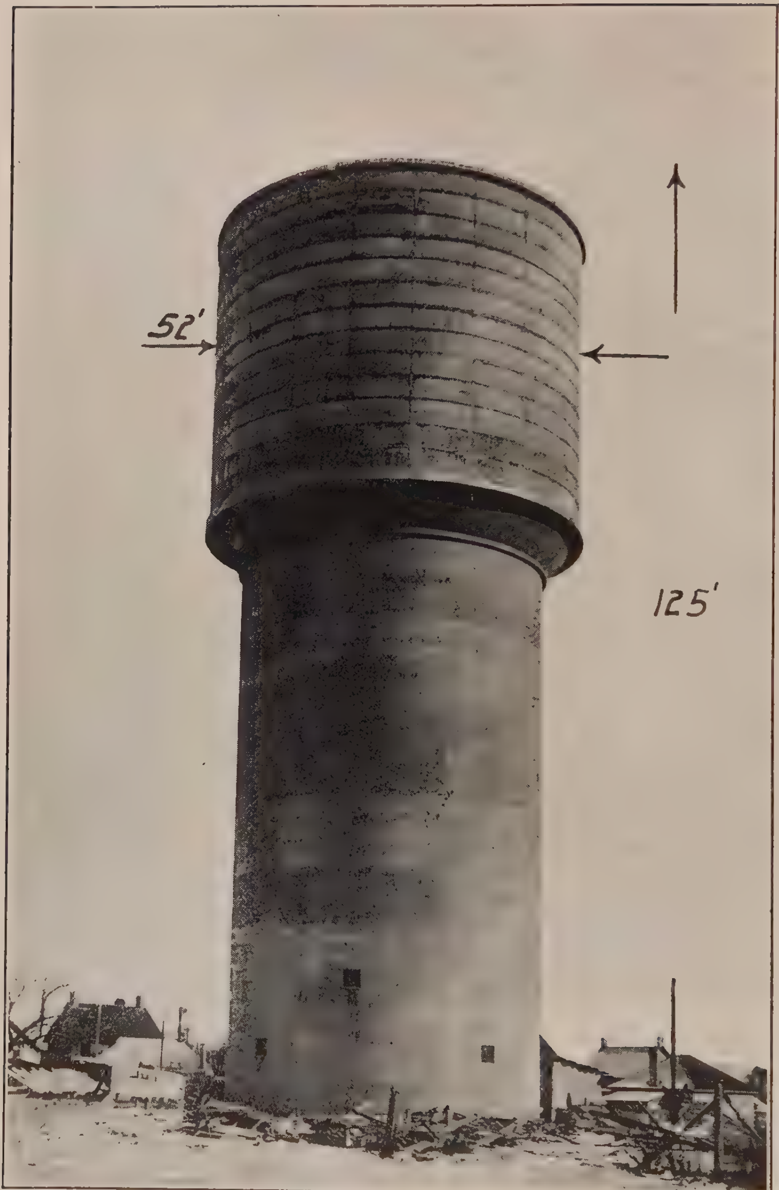

Fig. 4. 600,000-gal. Water Tank of Unwaterproofed Reinforced Concrete, Berlin, Canada.

The water tank shown in Fig. 4 has a capacity of 600,000 gallons, and is of reinforced concrete, not waterproofed; it is the practice of the builder to increase the quantity of cement in the mixture as the pressure increases. For extremely high pressures, it is the practice to use a rich cement finish to increase the density of the exposed surface; the usual proportions for the concrete are 1-2-4.

Fig. 5. 5,000,000-gal. Unwaterproofed Reinforced Concrete Reservoir, Duluth, Minn. Leakage developed at a construction seam.

The possibility of making structures of this character attractive is illustrated by the water tower located at Fort Revere, Mass.; this encloses a standpipe 50 ft. high and 20 ft. in diameter, with a thickness varying from 7 in. at the base to 3 in. at the top. The entire structure was built of reinforced concrete without the use of waterproofing of any character. The above tanks are all practically waterproof and are representative of a great number of similar structures which are successfully constructed without the use of waterproofing.

As reservoirs increase in size, they take longer to build, and it is not always practical to pour the concrete continuously; construction joints are necessary and extreme care must be taken to prevent leakage, especially if there is a considerable head of water in the reservoir.

In Fig. 5 is shown the five-million-gallon reservoir at Duluth, Minn., in which no waterproofing was used. This was built for the city of Duluth, and I am advised by Mr. E. W. Kelly that during its construction the terrific storm of Novem-

Fig. 6. 2,000,000-gal. Reservoir, Philadelphia & Reading Rwy. Co., Yardley, Pa.; shows method of applying water-tight seal.

ber, 1913, which caused so much damage and loss of life on the lower lakes, occurred when the wall was still lacking 10 ft. of its total height; this caused an interruption of about 24 hours in pouring the concrete; the joint was at the top of the earth embankment, and the adhesion at this point was broken before the water was turned into the reservoir, presumably because of unequal expansion above and below the joint. The seepage is unimportant in amount, but by reason of the possibility of damage from frost, measures will probably be taken to make it water-tight.

It is a practice in the construction of reservoirs to insure water-tightness through the use of an elastic impervious seal placed between two layers of concrete. In Fig. 6 may be seen the two-million-gallon reservoir of the Philadelphia & Reading Railroad at Yardley, Pa., illustrating the method of filling joints, placing the felt, and waterproofing it with a bituminous coating.

Structures that are also required to be reasonably water-

Fig. 7. Hollow Reinforced Concrete Dam, Sheldon Springs, Vt. No waterproofing used; structure substantially water-tight.

tight are dams of the type shown in Fig. 7, which is a view of the lower side of the hollow dam built at Sheldon Springs, Vt.; no unusual precautions were taken to waterproof this structure and it is substantially water-tight.

Structures that are used to store water must be sufficiently tight to avoid loss by leakage. In contra-distinction to this is the dry dock in which the problem is to prevent the leakage of water into the dock. The dock in the United States Navy

Yard at League Island, Philadelphia, Pa., was built entirely of concrete without the use of waterproofing; it was not built as a monolith, and although fairly water-tight, there is a slight seepage as a consequence at construction seams; the seepage, however, is not sufficient to cause trouble.

In the construction of Dry Dock No. 4 at the Navy Yard in Brooklyn, N. Y. (Fig. 8), every effort was made to make it water-tight; the concrete was proportioned and mixed for maximum density, reinforced against shrinkage and temperature,

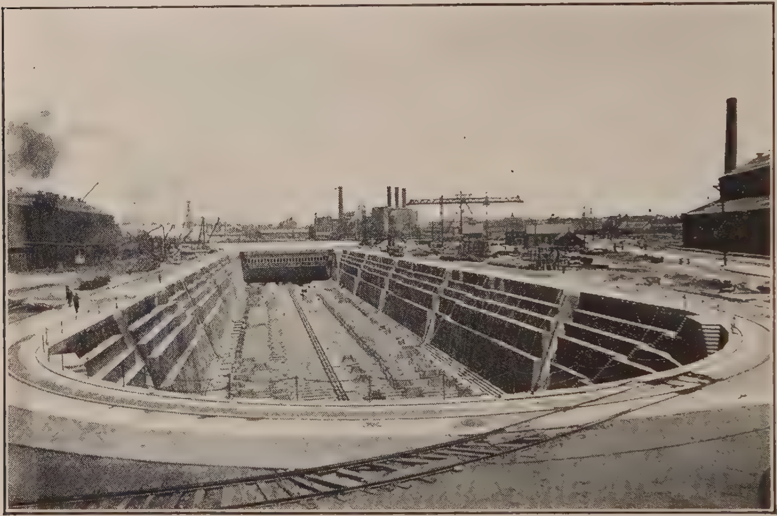

Fig. 8. Concrete Dry Dock No. 4, U. S. Navy Yard, Brooklyn, N. Y. Waterproofing material incorporated in the concrete; not entirely water-tight, there being some slight weeps.

and a waterproofing material was used in the mixture; the dock has slight weeps which are not considered objectionable. The difference in water-tightness of the two structures is not sufficiently important, in the writer's opinion, to warrant the additional expense of the waterproofing material that was used.

The dry-dock walls are of considerable thickness, and the difficulty is not in making the concrete water-tight, but in carrying on the construction so as to avoid seams and cracks through which leakage may occur. Small cracks become tight

as they are filled with sediment deposited by the water; the leakage from such cracks is usually not sufficient to occasion serious trouble.

Ordinary conduits under low pressure are not difficult to build water-tight. As the pressure increases, however, the difficulty increases; and for heads of upwards of 100 ft., considerable skill is required to build a conduit, either continuously or in sections, and avoid leakage.

The type of continuously-poured concrete pressure pipe

Fig. 9. Reinforced Concrete Pressure Pipe, Cottonwood Cañon, near Roosevelt, Ariz. Pipe not waterproofed; substantially water-tight.

shown in Fig. 9 is located in the Cottonwood Cañon near Roosevelt, Ariz.; this pipe is built of reinforced concrete without waterproofing, and is 5 ft. 3 in. in diameter, with 7-in. walls; the head varies from 34 to 74 ft. and the pipe is water-tight. It should be noted that the Salt River carries a great deal of sediment (sometimes as high as 4 or 5%), which is favorable to the development of water-tight structures.

For heads of upwards of 100 ft., it is a question whether it is cheaper to build of reinforced concrete or of steel.

Where the conduit is imbedded in earth, the changes in temperature are not so great, and the formation of cracks is thereby greatly retarded. If the pipe is wholly or partly exposed, the problem of design is not only one of caring for the stresses due to the head of water, but also for those due to temperature. It is common practice to build concrete pipe in sections which are allowed to thoroughly harden before being

Fig. 10. Reinforced Concrete Sewer Pipe used by City of Philadelphia, Pa. Pipe substantially water-tight under 10-16 lb. pressure.

placed in position, and a water-tight joint is made of some elastic, water-tight material.

In ordinary conduits and sewers such joints are usually made with cement. As the pressure increases, however, the difficulty in making these joints water-tight increases.

The pipes shown in Fig. 10 are used by the city of Philadelphia, Pa., in the construction of sewers; they are made with cement joints and are found satisfactorily water-tight under a pressure of 10 pounds. The larger size sewers are built of rein-

forced concrete cast in place. An illustration of this type is the outlet of the Wakeling Street main sewer shown in Fig. 11, which is considered in every way satisfactory from the view-point of water-tightness. The small leaks that develop in sewers are generally closed through the seepage of water carrying with it finely divided material.

The Queen Lane water filtration beds were built of reinforced concrete, constructed in the manner shown in Fig. 12.

Fig. 11. Reinforced Concrete Outlet Section, Wakeling Street Main Sewer. No waterproofing was used in this construction.

This work was built in sections which resulted in a construction joint at the top of the arch, as indicated in the illustration; there is a small amount of leakage at these joints, in eighteen of the twenty-six filters, but it is not considered sufficient to warrant the trouble and expense necessary to make them water-tight; the leakage is believed to be decreasing. It should be noted that the mixture is 1-2½-5 and that no waterproofing was used. Except at the joints, as indicated, the concrete is

water-tight, and very satisfactory; a view of the under side of these arches is given in Fig. 13.

In the construction of subways, the question of water-tightness is one of prime importance, and care is taken to make them water-tight. The tunnel under the harbor, between Boston and East Boston, was built in 30-in. sections under a head of 70 ft. of water. It was found that while the sections themselves were water-tight, there was a slight leakage at the joints, which were closed by pumping grout in under pressure.

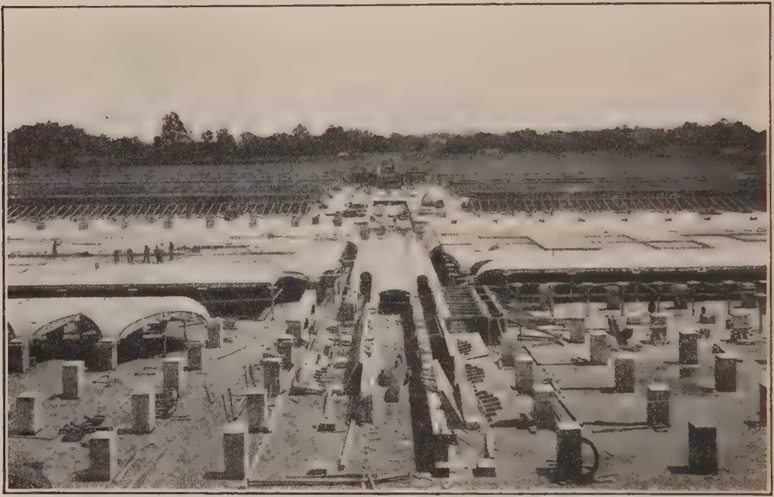

Fig. 12. Construction, Queen Lane Water Filtration Works, Philadelphia, Pa. This work was built in sections, necessitating construction joints, in which slight leakage occurred; remainder was water-tight.

In the construction of the filtration works at Little Falls, N. J., thin sections of 1-3-7 concrete were successfully built impervious to water, without the use of waterproofing; the concrete was faced with $1/4$ in. of 1-1 mortar and was water-tight under heads of 43 ft.

The inability to build subways continuously, necessitates construction in sections, and it is not easy to make the joints between them water-tight. The use of an impervious seal is therefore quite general.

In the construction of the east end of the Market Street subway in Philadelphia (Fig. 14), no attempt was made to

waterproof the walls, but the top was covered with a 1-in. layer of asphalt mastic.

In the construction of the New York subways, Mr. Alfred Craven, Chief of the Public Service Commission, states that "Our experience seems to indicate that with a reinforced concrete structure the waterproofing is not so necessary as with a steel beam type of section. In the former type leakage is apt to occur at lines where a day's work is ended. These have

Fig. 13. View of Groined Concrete Arches, Queen Lane Water Filtration Works, Philadelphia, Pa.

later stopped due probably to infiltration. With beam construction the leakage has been observed along the bent lines.

"In our latest construction, waterproofing is used on the roof at all open cut work; at stations where leakage may damage the finish work, waterproofing is added on the sides; where the structure is below ground waterproofing is also placed on the bottom and carried up on the sides to a point about two feet above ground water".

Concrete is being extensively used in solid floors for bridges, which must be water-tight to prevent dripping of ballast water; as the concrete is placed against the flat faces of steel, and it is not possible to make a bond between the concrete and steel that will be water-tight, leakage will occur and will increase as the structure expands and contracts; to prevent this leakage at these seams, it is the common practice to use some form of impervious seal over the entire top surface

Fig. 14. East Market Street Subway, Philadelphia, Pa. No waterproofing in side walls; roof covered with 1-in. layer of asphalt mastic.

of the floor. In this case, after the seal is completed, it is covered with reinforced concrete to preserve and protect it from injury.

By reason principally of the construction joints, it is also the practice to apply a similar treatment to the top of concrete bridges, as shown in Fig. 16. After the material is applied, a protective covering of brick, sand or concrete is placed upon it. It is essential in this method of treatment that there

shall be ample drainage, as the presence of water greatly reduces the life of the material.

The view in Fig. 17 is the swimming pool of the Detroit Athletic Club, and illustrates the application of an elastic seal extending continuously over the floor and pool.

Examples may be multiplied indefinitely, but in the above the writer has tried to illustrate the principal applications of waterproof concrete.

Fig. 15. Subway, 149th Street and Morris Avenue, New York, N. Y.; shows method of applying waterproofing.

VII. WATERPROOFING STRUCTURES THAT LEAK.

The problem of rendering existing structures water-tight is often difficult by reason of inability to locate the source of leakage, which is frequently a considerable distance away from the outlet. If the structure has been waterproofed with an impervious seal, it may be necessary to tear it out and replace it, which is an expensive operation. If the structure is of concrete, leakage will generally be at the cracks or construction joints, and it is possible to fill these with some bituminous material and thereby make them water-tight. Cracks

are also frequently closed by pumping in grout under pressure. If the leakage is not particularly great, it may in time heal itself by filling up with fine material.

The conditions, under which leakage develops, vary so widely, that it is impossible to lay down any method of treatment, and it is necessary to consider each case separately.

Fig. 16. Concrete Arches, Boston & Maine Railroad, Lynn, Mass.; shows method of applying waterproof seal.

VIII. EFFECT OF DESTRUCTIVE AGENCIES ON WATERPROOF CONCRETE.

Waterproof concrete, when properly used, is one of the best materials to resist the action of frost, alkalies, sea water, etc.; its resistance to electrolysis is far greater than that of any other material of construction.

Aside from its utility—which may be such as to necessarily limit its life, due to insufficient capacity or strength, or to unsuitableness for changed conditions of service necessitating its removal for improvements—a structure of concrete is subject, in whole or in part, to the action of the elements. It is,

therefore, essential that it shall be built to afford the maximum of resistance to these agencies.

The destructive action of alkalies is similar to that of frost, and results from the penetration into the pores of the mortar, or concrete, alkali-charged moisture, and the rapid crystallization of these salts due to rapid evaporation of the water. Con-

Fig. 17. Swimming pool, Detroit Athletic Club, Detroit, Mich.; shows method of applying waterproof seal.

crete, to resist such action, must be waterproof and thoroughly hardened.

The action of sea water on concrete is also the result of permeability, and will not, therefore, be likely to occur if the concrete is dense and impervious.

It is found by experiment that those concretes which are dense and have low surface absorption offer the greatest resistance to the action of alkalies and sea water; and experience proves that concrete can be so proportioned, mixed and placed as to efficiently meet these conditions.

Another problem that necessitates dry, water-tight concrete is that of preventing electrolysis. Experiments show that dry concrete affords the greatest possible resistance to the passage of electrical currents, but it is absolutely essential that it shall have the least possible absorption. The problem, however, is not a difficult one, since concrete can be made of the density requisite to insure dryness.

Where the structure is along the seashore, and is subject to the action of salt air, it is necessary that due care shall be taken to avoid the corrosion of the embedded reinforcement. It is necessary that there shall be no direct connection between the surface of the concrete exposed to salt air and the reinforcement. It frequently happens that a metal tie of some kind is used in construction, connecting the surface with the reinforcement. The writer has seen such corrosion of the reinforcement as to seriously disrupt the concrete.

Precaution should be taken under such conditions to avoid the use of highly porous aggregates, or those having a very high absorption. Such aggregates absorb moisture from the salt air, which moisture penetrates the mass to the reinforcement and causes corrosion. One of the Atlantic City piers is an example of salt-air corrosion resulting from an exposed metallic connection to the reinforcement, and of the presence of pieces of aggregates having a high absorption. In this case, the corrosion is so extensive that the stability of the structure was so seriously threatened as to necessitate repairs—and it is the writer's belief that the structure is still in an unsafe condition.

Similar conditions are likely to be encountered in some industrial plants, as, for example, abattoirs and meat packing houses, where the moist air, laden with brine, produces conditions favorable to corrosion. It is necessary, therefore, that the concrete shall be composed of dense aggregates, shall be of maximum density in order that it may have a low absorption, and there shall be no exposure to these corroding conditions of the reinforcement, or of metallic connections between the reinforcement and the exposed surface.

The practice of imbedding anchor bolts and other metallic

connections for the mechanical equipment of the plant, should be avoided.

A problem on which there is considerable uncertainty is that which is the subject of one of the papers before this Congress, namely, "Probable and Presumptive Life of Concrete Structures Made with Modern Cements". While those who have had an opportunity of examining structures in various parts of the world are confident that one of concrete, properly designed and constructed, is indestructible, this feeling is not shared by some engineers, because of the destructive action that has in some cases resulted from an improper use of concrete.

Since concrete structures which have thoroughly dried out, and from which water is excluded, are practically immune to the action of the elements, it is essential that they shall be perfectly dry and waterproof.

The problem of making concrete waterproof is, therefore, a question of vital importance, since its permanence depends on its successful resistance of the destructive agencies.

IX. FINAL REMARKS.

In conclusion, the writer wishes to emphasize the fact that concrete can be easily made waterproof without the addition of any foreign material, whether mixed with the concrete or applied as a coating to the surface, neither of which methods will prevent leakage if cracks develop.

Where it is not possible to pour the concrete continuously, and where construction joints are necessary, either particular care must be taken to make them watertight, or else recourse must be had to an elastic seal which will keep the water from the concrete.

The best method of making concrete waterproof is to properly proportion and grade the aggregates and thoroughly mix them with an adequate amount of cement; cement is the most efficient method of waterproofing concrete.

Waterproofing costs money, and it is a question whether the same care and expenditure of money, requisite to secure successful results in the application of waterproofing, applied

to concrete structures without waterproofing would not prove more economical.

The writer has always felt that the success or failure to secure waterproof concrete was due to the lack of sufficient preparation and care in erection, and that if the concrete was proportioned, mixed and placed with the same care that is required for successful results in the erection of other structures, the result could not be other than satisfactory.

The many successful and varied applications of concrete to structures that are water-tight serve to sufficiently demonstrate the fact that failure to secure such waterproof structures of concrete is due to an improper application of the material.

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(e) Journal of Electricity, Power and Gas.

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(f) In addition to the above there are numerous articles relating to waterproof concrete and waterproofing in the following magazines:

1. Cement.
2. Cement Age.
3. Cement Era.
4. Cement and Engineering News.
5. Cement World.
6. Concrete.
7. Concrete-Cement Age.
8. Rock Products.
9. Structural Conservation.

Paper No. 105

USE OF WOOD AND CONCRETE IN STRUCTURES STANDING IN SEA WATER.

By

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WOODEN PILES IN SEA-WATER STRUCTURES.

Propagation and Life of the Teredo.

Wooden piles standing in sea water are subjected to the ravages of the Teredo and the Limnoria. The teredo enters the pile between mean low tide and the mud line, through a hole of pinhead size and bores out its interior as it grows longer. The destruction wrought is thus unseen on the surface and wooden piles that have a safe look externally may be so honeycombed by the internal borings of teredo as to be absolutely incapable of bearing any load whatsoever.

The limnoria not only work more slowly but they attack the exterior of the pile and do not enter into it; the destruction wrought by this wood borer being readily observed. The result of limnoria attack is to give the pile an hour glass shape, the most serious destruction being at a point midway between high and low water. Thus its range of action is not so great, in piles of long length, as in the case of the teredo.

After the teredo has entered a pile it usually follows along the grain of the wood, riddling the interior of the pile by a series of holes about one-fourth inch in diameter. Though the worms keep clear of each other in making these holes, or bores, at times they leave a partition of only one-sixteenth inch between the different bores. If by chance the bore of one teredo breaks through into the bore of another, it has been authoritatively stated that the first teredo thereby ceases to be active and soon dies. In a number of instances it has been noticed that

sea borers will not attack new lumber if any old lumber is in the same vicinity for them to work upon. In Alaskan waters the time of attack seems to be dependent upon the possibility of old piles being in the same vicinity.

The ordinary growth of a Teredo navalis depends on the warmth of the water. Generally they are from six to twenty-four inches long with a diameter up to three-fourths inch. In the East Indian and China Seas the Teredo gigantia is said to grow to a size of six feet in length and three inches in diameter, such types existing in shallow water among the mango trees.

Being curious to learn how such soft worms as teredo could eat with apparent ease into wood, especially into such hard wood as ''Black Ironwood'', a well known and experienced English dock engineer obtained a large quantity of teredo and made a study of their construction. He says that ''if anyone will take hold of one of these worms just behind the head, pinching it firmly and compressing the back of the head, such treatment will cause two strong, sharp black teeth to protrude, of such sharpness that the blade of a pen knife can make no impression upon them''. He also states that the teeth are surrounded and hidden by an exceedingly strong sphincter muscle; that the supposed head of the worm, for they are acephalous, is a perfectly designed grab with sharp chisel teeth well adapted for the work of wood boring.

Without going too deeply into the natural history of the teredo a few facts as to their general nature, the different species, their propagation, growth and life will no doubt be of interest. As is well known, the teredo consists of a boring head with its two sharp teeth, and of a body or a mantle containing two siphons that constantly increase in length as the worm burrows into the timber. While the function of the head is to grind away the wood in front of it, the siphons provide a means of pumping in water through one and extruding the pulverized wood or pulp, as it were, through the other. The outer ends of the siphons pass out of the tail end of the teredo in two branches into the water outside the timber like two outlets to a river. Food is also taken in through the intake siphon, the excretions passing out through the discharge siphon.

While there are a number of different species of the teredo,

Prof. Charles P. Sigerfoos, who made an extensive study of the worm at Beaufort, North Carolina, during the summers of 1895 and 1896, says that of the three species identified, namely, Xylotrya gouldi Jeffreys, Teredo dilatata Spengler, and Teredo navalis Linnaeus, the first two were very abundant, Teredo navalis being found but rarely. He says that X. gouldi is the most numerous of all and is found everywhere, growing to a length of some two feet or more. Since the waters at Beaufort are very salty and the warm season is long, it would appear that such a condition is not favorable to the propagation of the Teredo navalis and that the harbor engineer errs when he blames the destruction of his dock piling, irrespective of its location, to Teredo navalis; Teredo gouldi being apparently the species that so universally exists.

While perhaps no extensive research has been made of the different species existing on the Pacific Coast of U. S. and in Alaska, from the results of Prof. Sigerfoos' investigation it would seem reasonable to suppose that these species are of both the navalis and gouldi types. Such worms are far more active in Southeastern Alaska than they are farther south in the waters of Puget Sound. Though the teredo is very active in the cold waters of our Pacific Coast, throughout the eastern front of New England north of Cape Cod, where the waters are of the cold Labrador stream, it has been authoritatively stated that no trouble is had from teredo in using green lumber in sea-water structures; two contradictory conditions as respects the activity of the worm.

In regard to the propagation of the teredo, Prof. Sigerfoos states that "Some species of sea worms, including Teredo navalis, retain their eggs in the gills during their embryonic development". In case of Teredo dilatata he says that "The eggs and spermatoza when matured are extruded from the anal or exhalent siphon in a slow steady stream, which continues as long as the reproductive organs contain ripe sexual products", the eggs thus being "laid free into and fertilized in the water". In the case of the X. gouldi the same authority says that he rarely discovered such an extrusion, but can give no explanation for the difference in the habits of the two species. In one instance Prof. Sigerfoos estimated that a large female Teredo

dilatata laid one hundred million eggs, which would seem to account for their multitudinous numbers in the oceans of the world.

The eggs of the species "that lay their eggs into the water" are developed very rapidly "and on warm days the embryos become free-swimming within three hours after the eggs are laid. Within a day the shell has been formed and the typical lamellibranch veliger stage reached". The mode of life of the larva and the rate of its development after hatching from the egg and becoming free-swimming Prof. Sigerfoos says is very difficult to surmise. Though the period of free-swimming mode of life which the larva of the marine lamellibranch enjoys before attaching itself to submerged lumber must be at least a month, Prof. Sigerfoos was unable to find any species undergoing this development.

During the period of "its free mode of life" a bivalve shell will be developed, into which "the whole creature can be withdrawn for protection" like a clam; also "a large swimming organ, the velum, by means of which the larva swims freely in the water"; likewise "a long powerful foot, by means of which it crawls actively over surfaces; and the internal organization peculiar to advanced lamellibranch larvae"—a typical small bivalve possessed of a swimming organ.

At Beaufort, Prof. Sigerfoos states that these minute bivalves could, throughout the summer, be seen "crawling actively over the surfaces of submerged, unprotected wooden structures," having "just settled upon the wood" and moving "rapidly in search of a favorable place for attachment"; usually "some minute depression or crevice in the wood, though it may become attached to a perfectly smooth surface". Soon after attaching itself to the timber by means of a single long lyssus thread, the bivalve looses its velum and hence its power of leading a "free-swimming life". The larva immediately "begins to clean a place for its burrow by scraping away the surface of the wood with the ventral edges of its shell valves", the small particles of wood and other substances thus collected cementing themselves together over the larva so as to form a protective cover of conical shape. Thereafter "the further transformation of the larva into a small ship-worm begins and progresses

rapidly'', the foot becoming a pestle-shape organ ''which assists the shell in burrowing''. The opening of the shell at the ventral side is changed through a rapid development of knobs on the ventral and dorsal portions of both valves, so that their swing upon each other is at right angles to their former direction. Due to rapid development, the shell gapes at both anterior and posterior ends, so as to permit of the protrusion of the foot in front and of the siphons behind. ''The first row of small teeth forms on the external surface of the valves at the anterior edges and later on becomes the mechanical agent by which the worm bores into the wood''. This transformation Prof. Sigerfoos states takes place inside of two days from the time of attachment.

One of the most debated questions in natural history has been as to how the teredo burrows his way into wood. Some eminent authorities have attributed it to some chemical solvent that assists the mechanical action by softening of the wood. Another authority seemed to think that ''there were siliceous particles in the mantle to do the work''. While other solutions of the problems have been offered, it appears that Osler in 1826 was the first to suspect the real mode of operation, though he did not actually observe it. Prof. Sigerfoos, who actually observed the process under a microscope, states that the mode of burrow is as follows: ''While the foot performs a cupping action, thus drawing the shell close against the surface of the wood, the powerful posterior adductor muscle contracts so that the teeth (which are on the anterior edges of the valves and which point outward and backward) on the shell rasp away the wood. The valves are brought to their original position by the small anterior adductor. The comparatively very large posterior adductor is therefore the actual agent that does the work aided by the foot; the shell with its teeth is the tool with which the work is done''.

During the progress of burrowing, a constant stream of water is passing through the siphon system, carrying away the rasped off pieces of wood. Though Prof. Sigerfoos seems to think that some nutrition is obtained from the wood pulp, the main source of the teredo food supply, he says, consists of diatoms and algae which float in the sea water and are taken in through the siphon

system. Prof. Sigerfoos is also of the opinion that the burrowing action is a periodic function, alternating perhaps with the more active ingestion of food.

Activity of Teredo and Limnoria.

The most pernicious action of teredo and similar sea worms seems to be where the water is calm, salty and warm, with a degree of clearness, free from mud and heavy pollution. In some harbors sewage has so polluted the sea water that teredo long since ceased to exist therein. While such a condition has for a great many years existed in the waters immediately surrounding New York Harbor dock front, north of a line drawn from Bay Ridge to the eastern entrance of the Kill Van Kull, south of such an imaginary line and further out beyond the Narrows the teredo still exists in considerable quantities. In at least one instance in lower New York Harbor it was found that while sewage had so polluted the water as to act as a protection to green piling against teredo attack through tidal ranges, yet for a distance of from one to two feet above the mud line the teredo action was quite severe, thus showing that while teredo may not exist at tidal levels, such a fact is no guarantee that they do not exist near the mud line. the location of their most active attack. At the mouth of fresh-water rivers, the inflowing salt water of neap tide, being of a greater specific gravity than the fresh water of the river itself, has, in a number of instances, been known to carry the teredo along the bottom of the river further up stream than would otherwise have been supposed and the green piles have failed because of such a transfer of the teredo beyond the limits of true salt water conditions.

In the opening up of a new waterway at one of the Pacific Coast cities, a rather peculiar situation was created. Hitherto the winding stream had been free from teredo to a point quite near its out-flow into the salt waters of the adjoining sound. After the new straight channel had been cut through the territory, green pile structures that had hitherto been immune from attack were soon destroyed. Investigation showed that a layer of sea water, practically two feet deep and infested with teredo, existed throughout part of the new channel along its bed, though the water at the surface was still that of the fresh-

water stream. In view of the salting of Lake Miraflores at the Canal Zone, Panama, and the expectation that similar conditions will exist at Salmon Bay, Puget Sound, Washington, due to the lock recently completed at the outlet of the Bay into the Sound, a new problem seems to present itself to the harbor engineers in respect to the use of green wooden piles in what would ordinarily be supposed to be fresh-water bodies. The existence of a layer of salt water, infested with teredos, along the bed of supposedly fresh-water streams, is one which will no doubt be carefully studied hereafter by the dock engineer when constructing wooden pile structures under such conditions.

In ports situated on fresh water rivers far from the sea, such as Philadelphia, Portland, Ore., and New Orleans, wooden-pile docks are naturally free from the destructive teredo, though still exposed to other destructive actions.

While a few creosoted-pile dock structures have been built at Philadelphia the adoption of this type of construction was not due to teredo action, since the worm has never been known to exist nearer than 50 miles below the city, Philadelphia being situated on the Delaware River some 80 to 90 miles from the open sea.

In some of our southern ports, such as Jacksonville, Florida, situated some 27 miles from the ocean on a river at times brackish, it is said that decay due to weather conditions is far more serious than teredo action.

In the port of New Orleans creosoted pine piles, with a sixteen-pound treatment, are used almost exclusively. While such piles are not subject to attack from teredo or other species of wood worms (such things not existing in the fresh waters of the Mississippi at this point), they are subject to a rather serious decay due to a peculiar cause. From information furnished by the chief engineer of the Port of New Orleans, it appears that "A great many of the creosoted piles (in New Orleans docks) have begun to decay, due to the admission of moisture where the piles have been cut off to receive the caps after driving". While they have attempted to overcome this difficulty by applying a coat of creosote to the cuts, such a procedure does not seem to have proven very successful.

It has already been stated that along the eastern coast of

New England, where the waters are from a branch of the Labrador stream, teredo action is not so severe as it is south of Cape Cod, thus permitting green oak piles to be used in dock work at Boston, Portland and other Maine harbors. While there are isolated records of the thorough honeycombing of timber by teredo along the Maine coast, there have been cases where green oak piles "believed to have been submerged nearly 300 years" were found upon examination to be perfectly sound. Still it is well to note that in Boston Harbor "Limnoria Terebraus" is very active and destructive, operating from about one-half tide down to the mud line.

In the removal of an old wooden pile structure at Halifax, N. S., a year or so ago, it has been stated by the engineer in charge that no signs whatsoever of the Teredo navalis were found on any of the old piles pulled up and it would appear that said harbor was practically free from the attentions of the destructive teredo. On the other hand there was evidence of the activity of limnoria, the old piles having been gradually eaten through by this worm. The range of the damage extended from about two feet above extreme low water down to the mud line, even in a depth of thirty to thirty-five feet of water, the greatest activity taking place at about low-water level. The worms seem to have attacked even creosoted piling which was but four years old, to the extent of eating into some of the piles to a depth of one-half inch. This, perhaps, was caused by insufficient penetration of the creosote oil, or the use of an oil of inferior quality.

In connection with a series of tests made by the U. S. Forest Products Laboratory upon the resistance of southern yellow pine against teredo attack at both Gulf and Pacific Coast ports, they report "That it is rather difficult to determine in general which does the more damage to treated wood, teredo or limnoria" and "That there seems to be little difference in the intensity of teredo attack at various depths, the specimens (of wood) at the surface being attacked to but a very slight degree less than those placed at the mud line".

A rather peculiar case of the existence of teredo in fresh water has been reported from Texas. A government snag boat built far inland of long leaf yellow pine was first put to work

in the Brazos River, Texas, "spending altogether two and a half months in tidal waters". The next three years were spent in the same river but beyond the range of tidal influence and in waters that showed "no signs of barnacle or teredo life". Upon being hauled out at the end of the period stated, the bottom planking were "found to be fairly well honeycombed by teredo, some of them being three-fourths inch in diameter, 18 inches long, all still alive and healthy". In reporting this instance, Mr. R. G. McGlove says, "It is possible that the gradual transfer of the Waco (the snag boat's name) from salt water to fresh, together with the natural saltness of the river, has been a factor in preserving the life of many of the teredo that got into the planks". As Mr. McGlove states that at mile 418 (that is 418 miles from the mouth of the stream, as the author interprets it) the Brazos River shows a high percentage of salt water, such an instance of the existence of teredo may not be so strange after all.

A very curious instance of the existence of live teredo above the float water-line recently came to note in repairing a ferry boat in Seattle Harbor. The boat in question is kept in almost constant service, as are most ferries. It appears that due to the peculiarity of the boat's construction a pocket existed some 8 ft. above the water level, into which the salt water was constantly fed when the wheels were in motion, which permitted teredo to exist in the planking at the pocket to its thorough honeycombing, which of course required its entire replacing.

In one instance in Eastern waters a dump scow which had been used in the vicinity of Boston and Portland was found to be so badly honeycombed with teredo that it was necessary to rebuild its entire bottom, southern pine having been used in the original construction. The results of an investigation undertaken to determine the cause of the infection, in view of the fact that the teredo does not generally exist in said waters, lead to the conclusion that the timber had become infested with teredo in southern waters: a rather peculiar case, which in connection with the case reported from the River Brazos goes to show that the teredo is a most peculiar worm as regards its propagation, its boring as well as its mode and the conditions governing its existence.

Probable Life of Green Piles Subject to Teredo Attack.

The time necessary for the destruction of a green pile by sea borers varies according to its location. In San Francisco and southern California ports, 2½ years is said to be the longest possible life of an untreated Douglas-fir pile when used under the most favorable conditions, but less than one year when the pile is subjected to abrasion, as with the fender piles. In fact it has been officially stated that a green fir pile will become so honeycombed in San Francisco Bay at the end of 8 months as to have absolutely no bearing capacity. While there are said to be records of from four to six years for untreated piles in said waters where no abrasion took place, it is fitting to point out that "100% of all untreated piles driven by one company in four of the Pacific Coast ports had to be replaced inside of seven years".

The life of a green pile along the South Atlantic and Gulf Coasts of the United States is said to be from eight to twenty-four months, but where the waters are very salty and in the hot season, but three months; hardly long enough to permit of a green pile structure being built.

In Alaska, native spruce piles have been completely destroyed inside of eighteen months. Still it is claimed that cotton-wood piles will last much longer in these waters, especially if the piles are unpeeled. In Seattle a green pile will last about twelve months.

While the Pacific Coast dock engineers have not perhaps full information upon this important subject, Mr. W. D. Faucette has furnished valuable data covering all the important ports situated upon the Atlantic and Gulf Coasts. (See table on following page.)

Different Kinds of Wood Used in Dock Piles.

Soft wood piles, such as pitch pine or Oregon pine, are quickly attacked by sea borers. Harder types of wood, such as the oak and teak, are not so freely attacked as are the woods of the pine family.

The best class of Jarrah, such as is grown in West Australia, will effectively resist attack of sea worms.

While black ironwood is hard, tough and of great strength, teredo easily attack it. Since it rapidly decays out of water, it is not suitable for dock work.

Sneeze wood is another hard and strong type of timber. Since it contains a pungent oil, its resisting power against attack of sea borers is excellent—almost as good as greenheart—but it has to be banded against splitting in driving.

Vera can also be depended upon to resist the teredo.

Blue gum contains an acid or poisonous quality that is said to render it immune against sea borers. Still blue-gum piles were entirely destroyed by teredo inside of five years at a southern California port.

Greenheart is the wood most relied upon by foreign dock engineers. It is generally proof against sea worms of all kinds, due to the poisonous oil it contains. It is a very tough and strong wood but liable to split when being driven, so that it has to be banded for almost its entire length to permit of driving.

Probable Life of Wooden Piles Standing in Water More or Less Infested by Sea Borers.

By W. D. Faucette.

Location	Distance from Gulf Stream Miles	First Attack	Shortest known time of failure
New York	123	1 to 2 years	8 years
Norfolk	115	Immediately	6 months
Charleston	55	3 months	18 months
Savannah	67	After 1st summer	24 months
Fernandina	63	One season	3 years
Miami	5	Very soon	7 months
Key West		3 months	3 to 4 years
Tampa		6 months	1 year
Pensacola		3 to 4 months	1 year
New Orleans		1 to 2 days	1 year
Galveston		1 to 2 months	6 months
San Juan, P. R.*			6 months
San Francisco*			8 months
Seattle*			11 to 12 months
Vancouver*			
Alaska*			11 to 13 months
La Guania*			18 months

It has been stated that greenheart piles have been severely attacked by teredo in such warm waters as the Caribbean Sea.

*Added by author.

Palmetto piles as used in Florida are said to be free from teredo attack.

Douglas fir as found in the immense forests of the states of Washington and Oregon is almost universally used in all the wooden-pile docks on the Pacific Coast of this country, even up to 125- to 130-foot length piles.

Some time ago attempts were made to use piles from the eucalyptus tree in dock work, but the results were far from satisfactory, either in New Zealand, Australia or San Francisco.

Preservation of Wood Piles.

While numerous methods have been tried to preserve green wooden piles from attack of teredo and other wood borers, the "creosote treatment" is the most efficient protection devised up to the present date. A system of pile protection practiced for years by the Dutch engineers was to drive iron or copper nails into their wooden structures at regular intervals, the rusting of the nails creating an iron coating to the timber that acted in the nature of a preventive against attacks of wood borers. Another system was to wrap the piles with burlap and give the whole a coat of hot tar. Piles so treated are known as "wrapped piles". While this system served its purpose for a time and a number of dock structures were built on "wrapped piles", its lasting qualities did not prove sufficiently effective and the system has given way to the creosote treatment. A method of pile protection formerly used to a large extent in San Francisco Harbor was to surround the wooden pile with a concrete shell, four or five different methods having been devised for so doing. Similar methods have been used in Puget Sound waters to protect the trestle piles of one of the local trolley lines across the tide flats. While other methods of pile protection have been used in the past, they have almost, if not entirely, yielded to the creosoted pile.

Though a very concise history of the development of the art of preservation of timber by Mr. F. D. Beal, creosote engineer of Portland, Oregon, is to be found in Vol. XII, No. 1, of the Proceedings of the Northwest Pacific Society of Engineers, it is fitting to briefly state the history of the modern art of creosoting, viz., the "Injection of heavy oil of coal tar" into the wood.

This process was perfected and patented by Mr. John Bethell in 1838 and consisted in "Impregnating the wood throughout with oil of tar and other bituminous matters containing creosote and also pyrolignite of iron, which holds more creosote in solution than any other watery menstruum". The first commercial use of the mode seems to have been made in 1865 for the Old Colony Railroad. Even then ten years elapsed before there was any extensive adoption of the art. In 1875 extensive creosote operations were undertaken in connection with the L. & N. Ry. Since that date the art has been more and more widely used, until at the present date there are at least 150 operating timber-treating plants in the United States, though the details of their treatment are not the same—the creosoting process being used by some, the zinc chloride method by others.

The general method of impregnating timber with creosote oil, though the mode varies in detail according to system of preservation used, is as follows: After the green timber is put into the cylinders, steam is admitted and maintained at the desired pressure until the wood is thoroughly sterilized and the sap liquefied. A vacuum is then created in the cylinders so as to vaporize the liquefied sap and withdraw it from the timber, the cylinders being kept hot during this vaporizing process. After the wood has become perfectly dry the cylinders are filled with creosote oil and kept under pressure until the desired penetration of the creosote is obtained, after which the timber is withdrawn from the cylinders and stored or shipped to destination. From 24 to 36 hours are usually needed for the proper application of the process. When the timber is thoroughly dried out or seasoned before placing in the cylinders, the vaporizing part of the process is omitted.

As long as the protecting creosoted coat or shell of a pile remains unbroken, the interior of the pile is protected, no matter how thin may be the protecting shell. Once the coating is pierced, no matter how small the opening, the pile's defenses are broken, the teredo enter upon their journey of destruction at this point of failure and the rest of the pile coating becomes useless. Such a breaking down of the defensive coating can be brought about by improper handling, by abrasion and wear due to floating objects, etc., which of course should be kept out from

under the dock if any consideration is to be had for the life of the piles and safety of the structure as a whole.

A cause of greater danger than the danger from teredo attack on creosoted piles, is the checking or opening up of the pile due to driving, which of course exposes non-treated surfaces to the worm and hence permits its early entrance into the pile, in spite of the protection offered by the treated surface.

In the process of creosoting, wooden piles lose considerable of their elasticity and the outer surface is more easily ruptured than the surface of an untreated pile. Thus a treated pile in dock work will snap more easily under a lateral blow or pressure than will a green pile unaffected by wood borers. Since teredo attacks are most active below the low-water line and the abrasion of fender piles is for the most part above this line, it would appear that in certain cases where only small vessels use the dock, that creosoted fender piles are preferable to green piles. Where the fender piles are subject to the blows of heavy steamers, a green pile should always be used, as it is the more economical.

The fact that a green pile has more compressive and transverse strength than a creosoted pile has been definitely proven; hence it is apparent that a creosoted pile cannot be loaded to such an extent as a green pile, under the same conditions. In a series of tests made under direction of H. B. MacFarland of the Atchison, Topeka and Santa Fe R. R., it was conclusively demonstrated that, in general, the strength of an Oregon fir pile subjected to the steaming process of creosoting has only two-thirds its original strength, the transverse strength being decreased 42%, the compressive strength perpendicular to the grain 27%, and the compressive strength parallel to the grain 27%.

In building docks, a large amount of rip-rap is often dumped in around the creosoted piles after the piles are driven. Is it not reasonable to suppose that rip-rip so deposited may injure the protective creosoted skin of some of the piles, so as to permit of an early ingress of the teredo? Then again, in driving new piles through such a rip-rap, will there not be the possibility that the part of the pile between mud line and the top of the rock fill will be injured by sliding contact with the rip-rap

so as to permit the early entrance of the teredo? The writer does not wish to imply that such is always the case, but it is a point well worth considering in repiling a dock through a rip-rap fill, quite aside from the other difficulties of driving a pile under such conditions. If the original voids in the rip-rap are so completely filled with sand or mud as to exclude the teredo from the section of the pile surrounded by rip-rap, no danger may occur from such an abscission of the protective skin; but does rip-rap become so filled?

Amount of Creosote Treatment Needed.

It has been stated that experience indicates that the amount of treatment needed in creosoting a pile "is inversely proportionate to the distance of the Gulf Stream away and inversely proportionate to its latitude". Such a ratio does not apply to the Pacific Coast section of the country, for self-evident reasons. Mr. W. D. Faucette, who has made an extensive study of the subject, recommends the following as the necessary amount of creosote treatment ample for the protection of wooden piling.

Location	Distance of Gulf Stream from coast	Pounds of oil recommended
Miami (Bay)	5	20–28
Fernandina (Entrance)	63	24
Savannah, Ga.	67	20
Charleston, S. C.	55	20–22
Wilmington, N. C.	56	20
Norfolk (Cape Henry)	115	20
Atlantic City, N. J.	119	16
New York (Sandy Hook)	123	12

On the Pacific Coast the universal practice is to use a 12-lb. treatment for commercial work but a 16- to 20-lb. treatment for government work (mostly 16-lb.) standing in sea water. At San Francisco they use a 12-lb. treatment, while at Los Angeles the city uses a 16-lb. treatment.

Life of Creosoted Piles.

As the strength of a chain is that of its weakest link, so the safe life of a dock structure is that of its piles. While records show that green piles used in building foundations have lasted for over a thousand years, no such record has ever been estab-

lished for dock piles. It is perfectly apparent that local conditions will determine the life of either creosoted or green piles. While the former may in certain cases last for twenty to forty years, it is not the part of safe or sane engineering to assume any such length of life for a dock pile standing in teredo-infested waters. In New York City the green piles used in municipal docks last from thirty to forty years, in most cases longer than the safe and economical life of the superstructure.

Creosoted piles driven in 1876 in connection with the foundations of a railroad bridge over East and West Pascagoula River, Miss., not far from the open Gulf, were found to be in such excellent condition in 1904 as to permit of their being used in the foundation work for a new bridge at the same place, and they are supposedly in use at the present date. Since large volumes of fresh water exist at times in the Pascagoula, this example is not, perhaps, a fair one to quote as showing the life of creosoted piles in salt and teredo-infested waters; the more so since it has been stated that at times even untreated piles have not been attacked by teredo at said location.

Another, and perhaps the best, illustration of the long life of creosoted piles in infested waters is the case of a creosoted-pile freight wharf at Oakland, California, where the piles are said to have been ''24 years in service with no renewals''.

Again at Santa Monica, California, creosoted piles which had been in use for twenty-two years, upon being removed were found to be in such good condition as to warrant their use over again in another structure.

It has been stated that in the North Sea, creosoted piles will last about thirty years, but only twenty-five years in the English Channel, due to the warmer water and the greater activity of the teredo in said waters.

While the reports indicate that under favorable conditions creosoted piles will last for twenty years in San Francisco Bay when the usual conditions which result from flotsam, abrasion and other kinds of wear and tear are taken into consideration, the life of a creosoted pile in San Francisco docks has been stated to be about twelve years. On the other hand the records of the Southern Pacific show that only 8%

of the creosoted piles in service for twenty years had to be replaced for causes other than mechanical breakage. It was expected that the rest of the piles would have a total life of thirty years.

It is perfectly evident from study of the record that local conditions, the activity of the worm, the general use to which the structure is put, the amount and carefulness of the creosote treatment and other causes of a local nature will govern the actual safe and useful life of a creosoted pile in any given locality. In general it may be said that a creosoted pile dock structure has a safe life of about fifteen years in the average teredo-infested American port. With piles thoroughly impregnated with the best creosote oil and care taken to avoid unnecessary abrasion of the pile structure, it is perhaps possible to get a life of twenty years, commercial treatment and use—not a scientific one—deciding the question.

Loads Upon Wooden Piles.

It is very evident that piles in docks, due to their greater unsupported length above the mud line, cannot be loaded as heavily as land foundation piles. It is also evident that the soft mud found in most harbors will afford but little lateral support to the pile and that its real unsupported length is, therefore, greater than the length of pile above the mud line. It has been stated by one leading authority that, in general, only the lower third of the penetration should be subtracted from the total length of a pile in determining its unsupported length, in other words, that the length of the pile above mud line plus two-thirds of its penetration should be taken as the pile length in figuring its strength as an unsupported column.

For figuring the allowable load upon a dock pile, the following formula has been suggested:

$$S = f\left[1 - \frac{L}{60\,D}\right]$$

Wherein S = Permissible stress per sq. in. at center of unsupported length.
L = Unsupported length in inches.
D = Diameter at center of the unsupported length in inches.
f = Working stress in outer fiber per sq. inch
600 lbs. for long-leaf pine or white oak
500 lbs. for Douglas fir or northern pine.

With the varying length of piles found in an ordinary dock, such refinement as figuring the permissible load for each and every pile may be questionable, since years of actual experience have more or less determined a proper loading, and hence spacing, of dock piles. It is perfectly evident that a large diameter pile can be loaded heavier than a small diameter pile and that a concrete pile will carry more than a wooden pile, the loads upon the latter varying as their size. Still the load imposed upon dock piles depends not so much upon the strength of the material of the pile as upon its penetration and the carrying power obtained from the skin friction and end bearing of the pile.

Though at times, and maybe very frequently, dock piles are subjected to very heavy loads, it is not the practice to figure them for loads much, if any, over 15 tons per pile, as provisions must be made for diminution of strength due to decay from natural causes. When used in dock work, a load of from ten to fifteen tons per pile is considered a fair allowance for wooden piles, that for concrete piles being determined by the size and length used. The standard practice of the Department of Docks and Ferries of New York City is to allow 15 tons per pile under main part of the dock and 12½ to 15 tons under columns of the superstructure, the dock loading being taken at 500 lb. per square foot. A standard practice in some ports is 50 sq. ft. of deck surface to each pile.

USE OF CONCRETE IN SEA-WATER STRUCTURES.*

General Consideration.

As stated by one eminent engineer "There seems to be more uncertainty in the minds of the engineering profession relative

* To avoid any misunderstanding of terms as used in this section the author desires to distinguish between (a) "concrete poured en-masse-in-situ" where provisions are made to prevent the sea water from coming in contact with the concrete until fully set, hardened and cured and (b) "concrete poured en-masse-in-situ" where the sea water is permitted to come in contact with the concrete almost immediately after the pouring or within such a short time that the greenness of the mass will permit of infiltration of the sea water into the concrete, with the accompanying washing away of the matrix upon the surface of the concrete.

As used in this paper "poured en-masse-in-situ" refers to concrete

to the behavior of concrete in salt water than on almost any other use of concrete, plain or reinforced * * *'', a statement that appears to be well substantiated by the discussions that have taken place upon the subject among various scientific bodies. Still a review of engineering literature indicates far less uncertainty in this matter on the part of the European engineer than on the part of his American contemporary. It is also clear that, in spite of some early failures of concrete structures standing in sea water in this country, deeper knowledge and the results obtained by a more scientific treatment of the subject (which have eliminated some if not all of the causes of early failures), have given American engineers in general far more faith in such a use of concrete than they once had. This is witnessed by the large and substantial structures now being yearly constructed by them even in water subjected to frost action.

While the ''Art of Concrete'' as practiced on land by some may be the use of the least amount of cement combined with the greatest amount of the coarser ingredients, placed in the forms in the shortest possible time, such practice will most surely result in disaster if applied to concrete structures standing in sea water. Then again the fact that a certain material will give excellent results when subjected to one set of conditions does not imply that the same material will act equally as well when subjected to another and entirely different set of conditions. Steel is simply cast iron chemically changed. Whereas the former is absolutely trustworthy for both tension and compression, woe betides the engineer who subjects a cast iron structure to tension stresses of magnitude! Steel may be steel, but the

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that has been thus poured but which is not allowed to become thoroughly hardened and cured before coming in contact with sea water.

The term ''a premoulded concrete'' does not necessarily imply that the concrete has been manufactured at one place and then driven or erected in another as in pile work.

As used in this paper ''premoulded air-cured concrete'' means concrete that has been allowed to become thoroughly hardened and cured before sea water is allowed in contact with it, irrespective of whether the concrete was poured en-masse-in-situ inside a coffer-dam or otherwise, or whether the concrete member was manufactured as an out and out premoulded structure, as in concrete pile work.

use of manganese steel under salt water conditions, as one set of specifications called for, would most certainly have resulted in future trouble, except for the fact that the error was discovered in time to permit of a correction of the specifications.

Chemists distinguish between (a) "slow-setting quick-hardening" and (b) "quick-setting slow-hardening" cements. In like manner both chemists and engineers must distinguish between cements and the other ingredients of concrete that are suitable for sea-water purposes and those which are not. Further still, they must distinguish between a mixture of the ingredients that will give a true watertight monolithic concrete and one that will give a semi-watertight concrete full of voids (be they small or large) especially in reinforced concrete construction, and more especially in waters subjected to frost action. And still further, the man in charge of the concrete work itself must be sufficiently well trained and posted in the art of concreting to be able to distinguish between concrete for street work and concrete for use in structures standing in sea water.

Though the rougher uses of concrete may call for little knowledge or skill, its use in structures in contact with sea water requires the most painstaking care and watchfulness on the part of the supervising engineer, his various inspectors, and the entire concrete crew. Not only must suitable materials be used, but they must be so mixed and placed that, with the proper amount of water added, the concrete will be impermeable throughout the entire mass: An impermeable concrete, made of proper and well graded material, properly mixed, handled and deposited, sufficiently cured before being allowed in contact with the sea water, is the cardinal feature to be provided for in using concrete under salt water conditions.

Absolutely impermeable mass concrete may not be so important in tropical climates, provided the materials used are suitable, but in reinforced-concrete construction it is of vast importance in order to protect the steel from corrosion. The formation of rust results in the expansion and cracking of the concrete, which will permit of still worse infiltration of the sea water and its further injurious action on the steel as well as the concrete. Impermeability is of vital importance in all kinds of concrete, mass as well as reinforced, in waters

subjected to frost action, and failure to secure it is sure to result in disaster.

During the past few years a series of microscopic studies of concrete have been made by Nathan C. Johnson at Sibley College, Cornell University. These investigations prove that even the best of concrete may be full of minute voids, caused, perhaps, by air bubbles incorporated into the concrete in mixing, and creating a capillary action, which, assisted by the phenomena of absorption and adsorption, may cause water to thoroughly penetrate the mass of permeable concrete, which may result in the disintegration of the entire structure when subjected to frost action; also a lack of uniform distribution of the ingredients as to cause bridge action throughout the entire mass, and not a mass glued together by thin layers of cement, the ideal condition of a perfect concrete, and that the concrete may contain a large amount of unhydrated cement. Unhydrated cement incorporated in a mass of concrete may become an element of serious danger; it is not only so much inert matter, doing no duty whatsoever except to fill the voids, but when brought into a hydrated state through the absorption of water by the porous concrete, may create new reactions, which in turn will tend to disrupt the concrete mass.

Cements.

In explaining past failures of concrete structures in sea water some engineers hold that the causes are mechanical; others, that they are of a chemical nature. While mechanical causes may predominate in freezing climates, in non-freezing waters it is evident that, where disintegration does occur, forces of nature other than frost action are at work, and these forces are, undoubtedly, of a chemical nature.

Cement is a chemical mixture of lime (CaO); silica (SiO_2); alumina (Al_2O_3); ferric oxide (Fe_2O_3); magnesia (MgO) and other ingredients mixed in a certain proportion so as to obtain the proper strength or the proper setting and hardening.

Chemists long since established the fact that magnesium and other sulphate contents of sea water had a disintegrating effect upon the aluminum compounds in cement, which tend to weaken and soften the concrete. It has also been established that the lime of the cement of fresh concrete into which sea water has

penetrated is apt to be (in part) withdrawn in the form of calcium salts by the magnesium salts of sea water, leaving a deposit of magnesia in place thereof. This magnesia, either alone or in mixture with the lime from the cement, constitutes the white substance found in the interstices of porous concrete between tides, and which will sooner or later result in the failure of the structure, concrete so affected having but little strength.

Actual experience has likewise demonstrated that the sulphate contents of sea water are most to be feared, since they act upon the unstable compounds of lime which form during the long period of the hardening (not the setting) of the concrete. Especially is this so during the first part of the hardening process when these lime compounds are least stable. Actual experience also goes to show that the most injury to concrete placed en-masse-in-situ in sea water takes place during the first part of the hardening period, and this points to a premoulded, air-cured concrete as the best for concrete when exposed to sea-water action.

As a result of the reaction that takes place between the magnesium and other sulphate contents of sea water and the calcium hydrates and alumina content of cement, not only is the concrete weakened at the point of attack, but crystals are formed, which in forming expand, causing a swelling and cracking of the concrete. This in itself results in thin fissures extending throughout the weakest part of the mass, viz., the matrix, and unless a true impermeable concrete is had, the new fissures permit of the ingress of additional water, which in turn results in the formation of new crystals and an enlargement of the original fissures, and perhaps in starting new ones. This formation of fissures in concrete has been absolutely proven to take place, and shows:

1. The necessity of using a cement in which chemical reactions will not take place.

2. The absolute necessity of impermeability to prevent the introduction of the water into the concrete.

To render a cement immune against attack of sea water several special brands of cement have been put on the market. One is the well known "Iron Ore cement" wherein iron ore, which the magnesium and other sulphates of sea water do not affect dele-

teriously, replaces in part the usual alumina content of the cement.

Another process is the addition of true pozzolana (a volcanic ash) or of an artificial pozzolana (blast furnace slag of a certain chemical composition) to Portland cement in the ratio of from 30 to 60% pozzolana, to from 70 to 40% of Portland cement. The reaction of such a mixture of pozzolana and cement results in a pozzolana-Portland hydraulic cement immune from sea-water attack.

With certain American Portland cements the formation on the exposed surface of the concrete of a film of lime carbonate acts as an armor against action of the magnesium sulphate of sea water. This film will only form when the concrete structure stands in comparatively still water. At Atlantic City and in California waters the formation of a coating of a gelatinous matter on concrete piles standing in sea water seems to have acted as an excellent protection against any deterioration of the concrete.

European countries bordering upon the Mediterranean are perhaps fortunate in having beds of a very uniform limestone which contains but little alumina and a large amount of silica, which make a slow setting cement of excellent quality, so that sea works in which such cements have been used have behaved admirably, even for a period of forty years, and become as compact as stone. It has been stated that the Tiel cement (made in this locality) has not over 2% aluminum and absolutely no free lime when used for sea water structures. Natural pozzolana is also very abundant in Italy, the Italian engineers being the first to make use of it for sea-water concrete, and they look upon the use of cement under such conditions with suspicion unless pozzolana is used.

In explanation of the action of pozzolana in cement, especially as regards the long life of the two sea moles built in the days of Nero and Hadrian, still in existence, the noted Italian engineer, Prof. Luigi Luiggi has stated: "Pozzolana contains a large amount of free gelatinous silica ready to combine with the free lime that unfortunately is always present in Portland cement, even the best. Thus when pozzolana was added in mixing Portland cement concrete, there was always present a large excess of

silica over this free lime, and the result was a kind of acid mixture instead of the basis mixture that was the consequence of the free lime in cement. This acid concrete resisted better the decomposition of the sulphate of magnesia of sea water which acted on the basic mortar of concrete, decomposing it and causing free or inert magnesia to be formed and the sulphate of lime, which had a tendency to crystallize and expand; the consequence of the expansion being disintegration of the concrete''.

While some contend that an impermeable concrete only is needed to resist sea-water action, tests made by Mr. J. M. O'Hara go to show that the concrete must not only be impermeable but it must be made of cement that possesses the inherent qualities to resist the disintegrating action of sea water, a statement that seems well substantiated by other authorities on the matter.

In closing this section of the paper it is fitting to state a few of the essential qualities of a cement suitable for sea-water structures.

1. Cement to be of the very best quality.
2. Of fine pulverization.
3. Thoroughly analyzed and tested for its chemical properties.
4. Low in percent of alumina.
5. High in percent of silica.
6. Free as possible from gypsum.
7. Absolutely free from ''Free Lime''.
8. Slow-setting—quick and uniform hardening.

Sand.

On one occasion it was found that a sand, which upon ordinary inspection seemed highly suitable for concrete purposes, failed absolutely when subjected in the form of briquettes to the ordinary cement tests—in fact, the briquettes hardly maintained their integrity while being placed in the clamps, before any load was applied. It is of importance that samples of the sand it is proposed to use in sea-water concrete should be subjected to a thorough analysis, both for their chemical composition as well as their mineral structure. Sand in which each grain is covered with a thin film of a complex form of tannic acid

(such as occasionally exists), is not suitable for concrete purposes, since the thin film prevents the cement from forming a union with the sand grains, though in some instances the cement may contain some form of alkali with which the objectionable film unites and thus permits of a union of the sand and cement.

While a sand must be clean, sharp and free from all foreign material, the grade used seems to have a material influence upon the resulting concrete—a hard, coarse grade giving the best results in sea-water concrete. Though a small percent of clay and loam is permitted in a sand used in ordinary land concrete work, the presence of such foreign material should not be allowed in a sand used in concrete exposed to sea-water action.

Gravel and Stone.

Since the ultimate strength of concrete cannot exceed that of the stone or gravel incorporated therein, it is of importance to use a rock whose crushing strength is high rather than low, if the best results are to be obtained. It is evident that a soft shale rock is entirely unsuited for concrete aggregate; also rocks or stones worn smooth by wave action, such as the small, round pebbles found in river beds or on the shores of the ocean—the latter perhaps covered with a thin, mossy film. While a concrete made of chips of hard granite would result in high strength, its cost, except in small quantities, would be prohibitive. When obtainable, the hard trap rock so universally used in New York City and vicinity gives far better results than any other kind of rock available. Not only does trap rock have a high compressive strength, but when crushed it has an exceedingly ragged surface, which presents the best possible condition for a strong adherence of the cement to the stone.

Irrespective of the strength of the gravel or stone used in concrete work, it is of importance that its mineral structure and chemical composition should be definitely known. The mechanical components of both the sand and gravel should also be carefully determined and modified, if necessary, by the addition of screened material, so as to produce the maximum density for concrete to be used under sea-water conditions.

Water.

General. Materials which have no cohesive or adhesive power—as cement, sand, gravel or stone—cannot bind themselves into a monolithic whole without the introduction of a substance which possesses a binding or gluing property. Flour by itself can be blown to the four winds, but when mixed with water becomes a strong paste. While a perfectly dry material will not in itself flow, the addition of water will set up a flow that becomes either a great danger, as in landslides after a heavy rain, or a powerful adjunct to the engineer, as in sluicing operations and in shooting concrete. It is the binding and lubricating property of water that makes it the real life and substance of concrete and which ties together the incoherent mass into a lubricated material, the crystallization of the lubricated mass resulting in the finished concrete product. Water, when mixed with the cement, forms a sort of mineral glue that coats the sand grains and stones so that they stick together.

Though water has no adhesive or sticking power, it possesses a strong surface tension, viz, capillary attraction, the power of this attraction, as well as the strength and proper placing of the concrete, depending upon the amount of water added to the incoherent material. The best results are obtained when the amount of water is such that each particle of the cement, sand and coarser aggregate is covered with a continuous film of water. Too little water or its uneven distribution will result in a non-continuous film; hence a diminution in its binding power. If too much water is used, the thickness of the water film will be such as to decrease its capillary attraction—the water in this case acting more as a void filler than as a binder, and when evaporated leaves a void in the concrete, resulting in a permeable mass into which sea water can later penetrate. Not only does too much water break down the film tension, but it causes a separation of the ingredients, the finer particles of the cement being set afloat and thus becoming separated from the mass. Since concrete derives its strength from the crystallization of the cement by means of water, the separation of the cement from the other ingredients prevents them from being bound together and reduces the strength of the mass as a whole. Not only is water needed

in the mixed ingredients to accomplish the above results, but it is also desirable to keep the green concrete in a moist, not wet, condition during curing in order to complete the growth of the crystal to its full maturity.

The water film should be built up by a careful addition of water and by such a distribution as to obtain a uniform lubrication of the mass as a whole, in order to obtain the maximum density. It has been stated that most porous concrete is the result of improper watering of the mass as a whole, the lubricating effect having been diminished or the cement particles set ''afloat''. The proper watering of concrete not only furnishes the maximum of lubrication effect but likewise produces the greatest binding power, ''these two forces tending of themselves to draw the particles together into a dense mass''.

The influence of water as respects the permeability of concrete was demonstrated by a series of tests made by Mr. Bamber in connection with the Aberdeen Dock failure. These tests clearly show that a plastic concrete has more impermeability than a semi-wet concrete and far more impermeability than a moist mixture.

Hot Water. While no importance, if any attention whatsoever, has perhaps hitherto been given to the effect upon the concrete mass by the temperature of the water (except to prevent freezing during the winter season) it appears from the experiments made by Mr. Johnson at Cornell that temperature of the water (distinguishing between hot and cold water) has a most marked effect upon the resulting concrete. Note has already been made of the large amount of unhydrated cement in concretes, new or old. It needs no argument to prove that the elimination of such a thing in concrete will result in great improvement both in its quality and strength and in economy in the amount of cement needed.

Though water is the only ingredient in concrete possessing a gluing property, due to its surface tension, it appears that when in reaction with the cement, cold water (with its strong surface tension) drives the particles of cement together into scattered groups rather than wetting the surfaces of each particle by itself. It also seems evident that a thin celluloid surface is apt to form on the exterior surface of each group of

the cement particles, which prevents the inside particles in the groups from obtaining water; hence, the existence of unhydrated cement in concrete. The breaking up of these groups of cement particles and permitting of free access of water to each of the cement particles in order to remove initial imperfections in the concrete, and thus obtain better results throughout the entire mass in the mixing process, is of vast importance, especially so in concrete used in sea-water structures. Too much care cannot be taken in obtaining the best of results in such undertakings.

The experiments made at Cornell go to prove that a liquid of low surface tension will bring about the desired breaking-up process of the cement groups. While liquids of a low boiling point, as alcohol, ether and ivory soap, have a low surface tension and will bring about the desired results when introduced into the water of the concrete, a far more practical solution of the problem was discovered by Mr. Johnson, viz., the use of hot water of moderate temperature. Hot water has a low surface tension and has apparently been proven to be a most satisfactory agent in creating a wide and general dispersion of the particles of cement, thus preventing any unhydrated cement in concrete and giving it more strength and endurance. While the heated mass will tend to take a quick set, such a thing can be prevented by stirring or shaking of the concrete until it has been put in its final place in the forms. Since liquids of low surface tension have small capillary power, they will flow more readily over a dry surface. Thus a hot-water concrete will not only be free of unhydrated cement but will flow more readily into the forms and around the reinforcement.

Salt Water. The feasibility of using salt water in place of fresh in the manufacturing of the concrete is one often discussed. Perhaps very few data are to be had upon this question, as it is seldom thought worth while to refer to it in publishing accounts of work done in concrete. In the construction of the famous Key West causeway (the cement used in the structure being imported from Germany) it has been stated that salt water was used in the making of all the concrete. As this structure has shown no signs of disintegration

up to date, it would appear that such a use of salt water has no injurious effect on the concrete.

Herr Blocq van Kuffeler, a noted engineer of Holland, has said that since the amount of salts introduced into concrete by diffusion when the concrete is exposed to sea water is vastly in excess of the amount that might be introduced into concrete "by the small quantity of water used in its manufacture—the use of salt water in the making of concrete is in all probability prejudicial to only a slight extent". Still, to be absolutely sure, it is wise perhaps to use fresh water when obtainable and thus be on the side of safety, with one less questionable point to contend against in the use of concrete in sea-water structures.

Waterproofing Compounds.

During recent years much has been said and published as regards the excellent results obtained in using some form of waterproofing compounds in concrete. There is no doubt but that some of the so-called waterproofing compounds are of considerable worth. On the other hand, a chemical analysis will easily demonstrate that others are absolutely without merit, simply commercial compounds masquerading under the name of a waterproofing material, "a little lower in price but equally as good".

If the work of the U. S. Bureau of Standards is of any weight, concrete can be made practically watertight up to a head of forty feet without the addition of any of the so-called integral waterproofing compounds. The small amount of such compounds used per batch has little, if any, effect on the impermeability of the concrete except for the extra care required in mixing in the compound. The addition of the compound will not compensate for a lean mixture, poor material and improper mixing and handling of the concrete. It states that an impermeable concrete can be obtained without the use of any integral compound if the same care is taken in the manufacture of the concrete as when the compound is added.

Since the principal action of a waterproofing compound is of a lubricating nature, a little extra cement and the use of hot water will accomplish the same results, especially if the concrete is mixed sufficiently long and to a plastic con-

sistency so as to permit the particles to flow into position without tamping; though, of course, the concrete should be well spaded so as to avoid pockets on its surface and to insure a perfect bond to the steel; also to work out of the mass any voids that may tend to exist therein due to the aerating of the concrete by dropping or to the absorption of air by the water in process of making, or perhaps to air bubbles in the water itself before it became a part of the concrete.

While it is impossible to make oiled concrete (internally applied) a success for sea-water structures, the application of coatings of animal, vegetable and mineral oils to the surfaces of concrete blocks on the part of Japanese engineers renders them almost impregnable against the incessant action of the sea and as lasting as the natural stone of which they are made.

A coating of tar has also been used by a prominent English engineer to overcome detrimental effects upon concrete piles above high-water level—no deterioration having taken place below said elevation—a fact somewhat contradictory to American experience.

Mixture and Mixing.

(a) General. In view of the high development in the art of manufacturing present-day cements there can be but little doubt that most failures of concrete in sea-water structures are due to an improper use of standard brands of cement, including improper proportioning, poor mixing, or both, and not to the cement itself. Especially does this seem to be true in view of the long-time and successful uses of concrete exposed to salt-water action where both the mixture and mixing were proper. A proper handling of a poor material will more often produce better results than an improper handling of first-class material. Thus the problem becomes a demand for a better concrete, not a better cement.

Concrete is simply an attempt to manufacture an artificial stone out of cement, sand, stone or gravel, and water. While the ingredients of a natural stone are far more compact than it is possible to make them in an artificial stone and the layers of the cementitious material are far thinner in the former product than in the latter, the more nearly the combination of materials used in making concrete approaches a natural

stone, the greater will be the strength and density of the artificial product.

(b) **Mixture.** The first requisite of concrete being strength, and strength being a function of density, it follows that an impermeable concrete gives both strength and density, these two functions existing in the same ratio in a properly mixed concrete.

The strength and density of concrete are limited by the density and strength of the weakest of its ingredients, usually the matrix. Not only is the matrix the weakest of the ingredients but it is the most expensive of them all. Thus the less the matrix over and above the amount actually needed, the better and more economical will be the concrete. Since the matrix acts as a glue, a stronger concrete will result when the adhesive mixture is uniformly distributed in a thin layer among the aggregate than when a thick film with an uneven distribution is had.

When the coarser aggregates are separated from each other by just a thin film of glue (as long as the gluing property is perfect throughout the mass), the greater the proportion of the coarser aggregate in the mass, the stronger and more dense will be the resulting composition; the best results being obtained when the surface area of the coarser aggregate is in proper proportion to the cement matrix—considered both as a void filler and as an adhesive agent. Since such a mixture of the artificial concrete product is very seldom, if ever, obtained in actual work and the stresses are transferred from aggregate to aggregate by the cement matrix, due to an ununiform distribution of the coarser aggregate, the burden of the load is transferred from the stone or gravel aggregate to the cement matrix. Thus the more perfect the matrix in its mixture, the stronger will be the resulting concrete.

The maximum density and hence the maximum strength of concrete is obtained when the amount of sand used is just enough to fill the voids between the stone, and the amount of cement incorporated into the batch is sufficient to coat the entire surface of the stone and the sand grains and to completely fill the voids between the sand grains. Such a mixture gives the ideal conditions for a perfect concrete.

The arbitrary methods hitherto used in proportioning the ingredients of concrete have been proven to give inferior results, both as respects strength and density.

It will no doubt be of value to cite here the practice of those countries which have met with the most success in using concrete in sea-water structures.

Early practice in England seems to have been a one part cement, two parts sand and three parts gravel, viz., 1-2-3 mixture.

Modern Holland practice indicates a one part cement, ½ part trass, 2 parts sand and 5 parts gravel, viz., 1-½-2-5 mixture.

Tests made by the Scandinavian Association of Portland Cement Manufacturers of over 10 years' duration in the Baltic Sea demonstrate that in using cement under sea-water conditions a matrix of a weaker nature than one part cement to two parts sand will give unsatisfactory results. Said tests also prove that the cement must not be excessively high in its percent of alumina.

The tests made by Mr. Johnson also prove that "a mixture of the cement and sand in a ratio of 1:2 gives sufficient cement to coat over and to fill between the grains of sand; whereas in a 1:3 mixture the voids between the sand grains are only partly filled with the cement and the whole concrete is apt to be a weak, faulty structure."

The usual Italian practice has been stated to be 100 to 200 kilograms of Portland cement, 1 part ordinary lime by volume, 2 parts pozzolana, 3 parts gravel or ballast. It is said that such a mixture will keep plastic for 6 to 8 hours and thus permit of its being forced into each and every part of the form by the hydrostatic pressure which results in a very dense mass; and that such a concrete will not suffer any damage after 12 to 20 hours, even from the wash of the waves; and that it gives almost as good results as ordinary concrete laid in the open for block work, etc.

In case of pile work, the Italians introduce more cement than above indicated, with the omission of the lime on account of the heavy blows used in driving the piles.

In introducing trass into concrete, Herr Blocq van Kuffeler has stated that a mixture of one part cement and ½ part trass and three parts sand will insure a concrete which will remain

in a perfect state of preservation when exposed to sea-water action if the concrete has hardened in a damp atmosphere before being placed in situ, but that it will not do so if the concrete is exposed to the infiltration of sea water immediately after manufacture. In the latter case a mixture of one part cement, ½ part trass and 2½ parts sand should be used.

In San Francisco harbor work, a mixture of 1-1¾-3¾, practically 1-2-4 is the standard practice.

Generally speaking, the most modern practice indicates a mixture ranging from: one part cement, 1½ parts sand, 3 parts gravel or stone (1-1½-3) to one part cement, two parts sand, three parts gravel or stone (1-2-3), all of which fully agrees with the results of both the Cornell and Scandinavian tests.

(c) **Mixing.** It is generally conceded that a longer time should be given in mixing of sea-water concrete than concrete for use in land structures. The writer is acquainted with two instances where the specification called for a rate of not over 12 batches per hour, equivalent to 5 minutes for mixing each batch. There can be no doubt but that such a rate, though perhaps excessively slow, will give better results and a more impermeable concrete than a 30-second rate, though the latter may be going to the other extreme. This question of the time used in mixing is one well worth the careful consideration of engineers engaged in the placing of concrete exposed to sea-water action, whether placed "en-masse-in-situ", or in the form of premoulded work, the former no doubt needing a longer mixing than the latter.

While the universal practice in mixing concrete is to deposit all the ingredients in the mixer at the same time, it is possible that a better concrete can be had for sea-water structures by following some regular order of placing the ingredients in the mixer; a question that may well be worth investigating.

Some authorities state that the cement and sand should first be mixed dry and then mixed with the coarser ingredients after said ingredients have been thoroughly wet down, the water being slowly added as the mixing continues until the proper consistency is obtained; that the addition of the coarser aggregate to the prepared matrix (cement, sand, and water) will result in a very poor bond between the two materials,

while the addition of the water after the three ingredients have been mixed together will tend to wash the cement from the larger particles or drown out the cement and fine sand. In all cases it is desirable that the sand should be damp and not bone dry to assist in the formation of capillary attraction, which is greater between the cement and the sand than in the coarser aggregate.

Inasmuch as a very minute amount of sugar will absolutely prevent a batch of cement from hardening, the utmost care should be taken by those in charge of the actual work to see that such refuse as chewing tobacco, scraps of lunch, spittal of those who chew tobacco, also urine and other organic compounds, all of which contain sugar of glucose and thus have a marked effect on the hardening of the concrete, do not get into the sand, gravel or stone individually or into the concrete collectively.

While other factors may enter into the question, the old saying of "Eternal vigilance and attention to details is the price of success" applies most forcibly to the use of concrete in structures exposed to sea-water action.

Failures and Frost Action.

While there have been several noted failures, and perhaps a number of less important ones, of concrete when used in sea-water structures, it cannot be denied that in actual numbers there have been more failures among land structures than among concrete structures standing in sea water.

One of the most noted failures of a concrete structure exposed to sea-water action was the case of the Aberdeen Dock, Scotland, in 1887, less than two years after its opening. Without entering into a long discussion of the facts of said failure, suffice it to say that investigation established beyond a shadow of a doubt that:

(1) Inferior cement was used in part of the work
(2) The sand was of too fine a grade
(3) The larger aggregate was of a poor quality
(4) The mixture was of too lean a nature
(5) And last of all, that the concrete was poorly mixed, handled and placed, in the light of our present-day knowledge of the art.

The partial failure of a United States Government dock in Boston, built 1899-1901, which began very soon after the completion of the last section of the structure, has created more uncertainty in the minds of American engineers in regard to the use of concrete in sea water than any other instance which has occurred in this country. When it is considered that the concrete in this structure was partly placed in the wet through an open-top bucket, that an attempt was made to protect its surface by a mortar plaster, and that the temperature of the outer part approaches zero almost every winter, the results observed are not to be wondered at in view of our present knowledge of the subject.

Other failures in early uses of concrete exposed to sea water —especially where poured en-masse-in-situ, in Boston Harbor and at other locations along the New England coast—have all been due to the same cause, a permeable concrete exposed to frost action. The fact that there have been few, if any, failures of concrete standing in non-freezing or tropical sea water seems to fully substantiate the generally accepted opinion as to the causes of such failures as are reported from Boston and other New England ports. More recent undertakings in concrete construction standing in sea water both at Boston and Aberdeen are said to be in an excellent state of preservation at the present date, especially so with ''premoulded air-cured'' structures.

From results obtained with a concrete monolith built on top of a breakwater at Sandy Bay, Mass., it would appear that a troweled surface, as used in sidewalk and cement floor work, was of great assistance in preventing deterioration of concrete exposed to sea water and frost action.

While the failures of concrete structures standing in sea water along the eastern front of New England have in almost every case taken place between tide levels, at Southampton, England, the failures reported have in almost every instance been above high-water level, the concrete below said level being in a satisfactory condition.

Another remarkable failure of sea-water concrete of recent date has been reported in connection with one of the dry docks at the Brooklyn Navy Yard, where the concrete is said to have been so disintegrated as to be removable with a pick and spade. It is perhaps unfortunate that no data have been furnished the

engineering profession as to the reasons for such failure, especially so as the City of New York, on the opposite side of the East River, has obtained most satisfactory results in the use of concrete in the construction of the extensive sea walls along her water front, during the past forty years.

Other reported failures are:

Portsmouth, N. H.—A government pier built in 1908 of mass and block concrete. Deteriorated in places at end of four years to a depth of 8 inches, between tides.

Atlantic City, where considerable disintegration of the concrete in an ocean pier has taken place due to chemical action.

Lower New York Harbor, information concerning which the author does not have.

Lynn, Mass.—A sea wall built en-masse-in-situ, the face of which became badly affected by the action of the sea and pounding of the waves, efforts to repair which with the cement-gun process are said to be questionable in places. Since the ratio of expansion of concrete is said to vary according to the amount of cement used, but little success can be expected in plastering a concrete wall in frost climates.

Liverpool, England.—The underside of a concrete-dock deck slab disintegrated from the dampness getting at the steel reinforcement, due to a permeable concrete and the steel bars being placed too close to the surface of the structure.

Southampton, England.—Disintegration of pre-moulded concrete piles above high-water level due to electrolysis in one structure, but to unexplainable causes in another, the concrete below high-water level being in a perfect state of preservation.

San Francisco.—Failures of concrete columns under a dock, mostly in connection with the so-called Holmes pile. Concrete poured around a wooden pile, in some of which, it has been stated, no kind of reinforcement was used. Concrete poured into receptacles from which the water had not previously been extracted resulting in alternate layers of gravel, sand and cement throughout the column, which naturally failed when the forms were destroyed by the teredo.

At Halifax a large concrete dock was but recently finished. Some method was needed to protect the concrete piles above low

water from frost action. It was proposed to use creosoted lumber around the piles, but it was soon discovered that the creosote oils had a deteriorating action on the Portland cement, especially on the green mortar of the Portland cement. To overcome this action it was necessary to coat the side of the planks next to the piles with pitch.

In other places in Nova Scotia vitrified salt-glazed tile blocks filled with concrete have been used to prevent any further destruction of concrete due to frost action. A granite face is often used to protect concrete from frost action between tides.

In connection with the concrete dock work of the Great Lakes it is customary to use an iron plate or a wooden covering, extending a few feet above and below water level, to protect the face of the concrete from abrasion by ice or frost action.

Since most, if not all, of the failures of concrete structures standing in salt water in this country appear to have been in sea waters subject to frost action, it is well to inquire as to the effect of frost action upon such structures when standing in fresh water. While perhaps such structures may be of but recent date and thus constructed in accordance with up-to-date practice, a few important facts can well be learned from them.

A concrete breakwater built at Burlington, on Lake Champlain, in 1904, does not seem to have shown any signs of disintegration, although exposed to heavy frost and ice action.

In St. Paul where the temperature goes as low as 40° below zero, a concrete wall built of 1-3-6 concrete, in connection with a lock in the river, is said to have shown no signs of deterioration due to frost action.

Whereas every failure of concrete when exposed to sea-water action is widely advertised and commented upon, such failures of concrete which may take place in structures standing in fresh water are apparently passed by as ordinary events and made good by the engineers in charge, without further comment.

When the vast number of concrete structures built on land, such as dams, viaducts, bridge piers, etc., are considered and the success apparently had with them, especially when built in sections of the country subject to very low temperatures during the winter months, it would seem to be evident that when thoroughly cured and hardened before being brought in contact

with water in freezing temperatures, concrete will successfully resist frost action. Such concrete structures are usually built in the spring, summer and fall, and thus become thoroughly cured, hardened and dried out before the period of frost comes. Hence their success. When built in cold weather special pains are taken to keep them dry and warm until thoroughly cured and hardened.

Long Time Uses and Premoulded Concrete.

Concrete was used by the Romans and Carthaginians in ancient times. There are said to be concrete structures constructed as sea moles in the days of Nero and Hadrian still in existence and doing service in Italy at the present time.

Mass concrete was used by the English engineers as far back as 1840. Since that date not only has a vast amount of mass concrete been successfully used by foreign engineers in their sea-water structures, but during the last 20 to 25 years reinforced concrete structures have been used extensively by all foreign countries in the way of massive docks, piers, graving docks, breakwaters, lighthouses in exposed positions, boats, barges, floating caissons, coast revetments and in almost every conceivable kind of structure fixed and floating, to the extent of enormous expenditures of money.

Without attempting to give a complete list of such structures a few of the earlier ones, built years ago, and which are still in existence and reported in excellent condition, may be noted.

In using concrete in sea-water structures it was recognized by foreign engineers as long ago as 1865 that when placed "en-masse-in-situ" concrete was apt to give uncertain results and that far better results were had with "pre-moulded, air-cured" concrete.

One of the first applications of the pre-moulded block system of concrete construction was in the building of 3200 feet of quay wall at Dublin, Ireland, in 1872, a structure said to be in excellent condition at the present date. Other massive concrete structures, such as sea walls, dry docks and breakwaters—scattered throughout Europe and not confined to any particular section—were built on the same system between 1878 and 1888 by foreign engineers and have likewise behaved admirably

during their existence and are still in a satisfactory condition.

In the removal of some 68,404 yards of concrete from a sea wall at Liverpool, no single instance was discovered of a decayed or shoddy concrete, the entire mass being in a perfect condition though exposed to sea-water action for 40 years. It is of interest to note that the original concrete was placed in the dry and no doubt sufficiently cured before allowed in contact with sea water.

An old foundation built on Alcatraz Island, San Francisco Bay, in 1867, of concrete does not show any signs of disintegration, though exposed to daily tidal action.

In the building of the long sea-wall around New York City, marked success has been obtained by a proper handling of the concrete. The first attempts, made in about 1871, with "en-masse-in-situ" methods, did not give satisfactory results. The pre-moulded block system was then adopted with marked success; so much so that it is officially stated that no signs of serious disintegration have been discovered during the 40 years of the existence of the wall, though it is subjected to severe frost and ice action. It is well to note that at the time the first part of this wall was constructed a finely ground cement was unknown. Hence the lasting qualities of the concrete are the more remarkable, and are no doubt explained by the 1-2 matrix used in obtaining a dense surface of the blocks, if not throughout the entire block.

In the building of breakwaters, concrete was extensively used by European engineers long before the art of reinforced concrete became known. In Italy such structures have been in existence for over 30 years without showing signs of disintegration. Concrete was used by English engineers as early as 1871 in breakwaters which are said to be still in existence and doing duty. A concrete-block breakwater built in India in 1885 showed no signs of failure at the end of 22 years.

A few of the earliest reinforced concrete structures standing in sea water may be cited:

It is generally admitted as an historic fact, that the first reinforced concrete pile was made and used in France in 1896 by the noted French engineer, M. Hennique.

The first concrete sheet-pilings driven in France in 1898 in connection with a sea-wall, are reported to be in as good condition today as when first built.

The first reinforced concrete dock appears to have been built in England in 1899 at Southampton. This dock is officially reported to be "practically as good as new and shows no signs of decay whatsoever" even at the present date.

Other reinforced-concrete structures at Southampton have not done so well, due in part (on one structure at least) to a permeable concrete, dropping of the stirrups and the formation of voids therefrom, and to the whole strength being somewhat light for the service to which the dock is put, the dock having no X bracing below the high-water level. As all the damage has taken place above the high-water level, while the concrete below said level is in perfect condition, this concrete structure, considered only as one exposed to action of sea water, may be said to have given excellent results.

While the use of reinforced concrete in dock work dates from 1900, it must be evident that as far as European countries are concerned, the results have been sufficiently satisfactory to justify the investment of enormous sums of money in this type of construction.

Such structures are also found in large numbers along our Atlantic, Gulf and Pacific Coasts, as well as on the Great Lakes; in Canadian waters, as well as in those of this country. At least 50 such docks have been built since 1906.

In view of the long-time knowledge of the merits of "pre-moulded air-cured" versus "cast-en-masse-in-situ" concrete for sea-water structures, it appears strange that the latter method has been so generally followed by American engineers, even up to the present date. For years the pre-moulded system of block concrete has been employed by foreign engineers in massive structures in sea water with marked success and there is no reason why all American engineers, as some apparently have done, should not obtain equally satisfactory results with pre-moulded piles, cylinders, floating concrete caissons and block work.

There is no doubt in the writer's mind that if the "pre-moulded air-cured" process of placing concrete had been used in

a number of cases where the concrete was poured "en-masse-in-situ" with no special provisions for its hardening before it was allowed to come in contact with sea water, the subsequent failure of the concrete would not have resulted.

NOTE.—The author desires to acknowledge his indebtedness to those who so kindly furnished him with local facts concerning certain ports of our coasts, as well as to those authors from whose works he has been able to assemble part of the facts given in this paper; to the technical magazines, scientific books, society papers and discussions, foreign as well as domestic, detailed reference to which is impossible on account of their great number. He is especially indebted to Mr. Frank D. Beal, Creosote Engineer, Portland, Ore.; to Mr. Charles A. Newhall, Chemical Engineer, Seattle, Wash.; and to Mr. Herman F. Tucker, Consulting Engineer, also of Seattle, for valuable and timely suggestions made in their respective reviews of the first drafts of the author's paper.

DISCUSSION

Mr. B. F. Cresson, Jr.,* M. Am. Soc. C. E. (by letter), stated that in New York Harbor, wooden piles which have been driven for many years have shown signs, when pulled, of having been attacked by the teredo, but in recent years the teredo has ceased to be active in the harbor proper. At St. George, Staten Island, in the Lower New York Bay, in Jamaica Bay, and along the coast where there is clean sea water, the borers are comparatively active; but the contamination of the harbor waters has practically eliminated the teredo. Mr. Cresson.

The most expensive pier structures built by the city of New York are the so-called Chelsea Piers, on the North River between Little West 12th Street and West 23d Street. These piers are built with ordinary timber piles without any treatment whatever, and it is not anticipated that they will be attacked. In South Brooklyn, on the easterly side of the Upper Bay, the city of New York has built several fine modern piers on timber piles, and these piles have not been treated. At St. George, however, the foundations for the municipal ferry terminal were placed on creosoted piles, as it was thought that there was sufficient clean water there to make the teredo active, particularly if the plans for the better disposal of sewage should be carried out in the future. New York City dumps its raw sewage directly into the rivers, without any screening or

* Ch. Engr., New Jersey Harbor Commission, Jersey City, N. J.

Mr. Cresson. purification whatever, and plans (by the Metropolitan Sewerage Commission and other commissions and the city departments) are being prepared, in order that the harbor waters may be less contaminated by sewage.

In New Jersey, the Passaic River, which empties into Newark Bay, has become so foul that a trunk sewer is now being built to convey the impure waters, which have been dumped into the Passaic River, across the marshes and empty them after screening into the Upper Bay, near Robbins Reef. It is interesting to note that some of the railroad companies with terminals on New York Harbor have made their recent pier construction and ferry constructions of creosoted timber, anticipating, perhaps, the time when the teredo may again become active.

With regard to the experience here of concrete in sea water, the New York City Dock Department has built a great length of concrete sea wall. The better part of this has been constructed either by levelling off the bottom or by constructing a pile and timber grillage and placing thereon heavy molded concrete blocks. These blocks weigh, some of them, many tons and have been molded at the yards of the New York Dock Department and conveyed to the site on scows and put into place by the 100-ton derrick owned by the New York Dock Department. Concrete walls built in this manner are usually faced and capped with granite and are of practically permanent construction. Walls that have been in place for more than 20 years have been removed and the concrete blocks show no sign whatever of deterioration. Concrete walls built in place show general results far less satisfactory than molded blocks. The face of the concrete soon exhibits signs of deterioration and need of repairs.

Mr. Cresson is strongly of the opinion that in designing concrete structures to be placed in sea water, subject to considerable variation in temperature, it is wiser to employ concrete blocks, rather than concrete molded in place, even if the greatest care is observed in placing the latter.

There has been practically no experience whatever with structures placed in New York Harbor where concrete piles have been used, but they are now being contemplated in the new piers which the Standard Oil Company are preparing to build at Constable Point on the Kill van Kull.

Mr. Jacobs. **Mr. J. L. Jacobs**,* M. Am. Soc. C. E. (by letter), said that in waters of the Gulf of Mexico the action of the teredo and limnoria on timber is very severe. Situated as it is, immediately adjacent to the great yellow pine timber belt of the Southern States, the use of timber in all dock structures is logical and therefore extensively practiced. All wharf structures in New Orleans, Galveston and Texas City, the three principal Gulf ports, are of this type of construction. In Galveston and Texas City the solid sheet-pile bulkhead is used with protecting apron to hold the slope; in New Orleans wharves about one hundred and fifty feet wide,

* Res. Mgr., James Stewart & Co., Inc., Houston, Tex.

Mr. Jacobs.

with very flat slopes underneath, are used. Creosoting is resorted to wholly for protection against marine borers.

In some work done for the Galveston Wharf Co. in 1912, 24 to 27 pounds of creosote were used per cubic foot of timber, the heavier treatment being insisted on by the owners for the exterior layer of sheet-piles in the bulkhead. To compensate for the inevitable decrease in strength of the timber, due to the prolonged treatment and possibility of scorching, the piles were increased to 16-inch diameter at the butt; and the bulkhead, which was normally constructed of two thicknesses of 4- by 12-inch material, was increased to three thicknesses, the exposed piece having 27-pound treatment, the middle piece 24-pound treatment, and the inside piece, in contact with the filling, being of green material. This heavy treatment is very unsatisfactory. It is very expensive, not only on account of the additional oil required, but also because the time required to inject the last 3 pounds of a 27-pound treatment is as great as that required for the last 10 pounds of a 24-pound treatment. The resulting timber is very brittle and cuts like cheese.

While many specifications call for long-leaf timber to be treated, the consensus of opinion is that better results are obtained with the more open and porous short-leaf timber.

In all other structures in Galveston, 24-pound treatment is specified. In structures at Texas City, 20-pound treatment was specified, though tests showed an average injection of 22 pounds for the job.

When the Galveston Bay Railroad bridge, connecting Galveston to the main land, was removed in 1912-1913, it was found that very little trouble was in evidence in piles which had been in the structure as far back as 1875, many of which by test only carried 12 to 18 pounds of oil per cubic foot; and those placed in the structure in 1895, carrying 24 pounds of creosote oil per cubic foot, were in perfect condition when removed and were subsequently used in other structures.

In New Orleans along the Mississippi River where the water is fresh the practice has been to treat the piles with 16 pounds of creosote oil injected under the specifications of the American Railway Engineering and Maintenance of Way Association. The super-structure is treated with 12 pounds. There are, however, no marine borers in the fresh water. The variation in level of the river is from 17 to 20 feet; so on the most recent structures the engineers attempted to use, for sake of economy, a method of treatment which would result in a lighter average treatment, but distribute the oil where it would do the most good. This method contemplated the careful removal of bark from the pile, so as to leave the inner skin intact, and to remove the inner skin only from such parts of the pile as would require the heavier treatment—in this case the upper 20 feet, as the lower section would always be submerged in fresh water. This pile was to have been introduced into a cylinder and treated to a refusal, which, as estimated for piles 50 to 70 feet in length with 20 feet skinned, would be only 10 to 12 pounds per cubic foot average, but would give a complete treatment to the exposed sec-

Mr. Jacobs.

tion. It was found to be impracticable to secure the timber peeled in this fashion, and this method was abandoned for a uniform treatment of 16 pounds per cubic foot.

Concrete has not been used on the Texas Coast for structures to any very great extent, but as far as can be observed, there has been no deterioration of such structures as the Galveston Sea Wall and the Galveston Causeway, due to the action of sea water.

Paper No. 106

THE OUTLOOK FOR IRON.

By

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The close of the nineteenth century produced an attitude of mind in many students of national affairs akin to that of a merchant who balances his books at the end of a twelve-month. When the results of a year's business have been demonstrated, the merchant decides on his plans and policies for the future. He makes a reliable estimate of his resources and learns his possibilities and his limitations. As a nation which looked over a hundred years instead of one year, we were in much the same position when the twentieth century opened.

From small beginnings, all manner of industries had reached an impressive development. Some employed materials which were constantly reproduced either by plants or animals, and which, by improved methods, could be increased in amount; but other industries were rapidly drawing upon fixed reserves which could not be renewed. We naturally began to forecast the future and, with a look ahead, to infer the course of events in the century then opening. Among the industries, that of mining came in for special attention. It is a very great one in this country, and it is distinctive in that it destroys its raw materials in utilizing them. Forests, crops and live stock all grow again. Ore and coal once mined are gone forever. Not unnaturally, in a fundamental industry such as iron mining—one on which so many others rest,—people vitally interested began to raise the question of reserves for the future and to wonder in what position the industry would find itself fifty or a hundred years later. We are not surprised, therefore, to note that open expression was given to feelings of apprehension, nor that some prophecies were made whose re-statement now possesses much interest. Not alone,

however, in our own country were these apprehensions felt. Abroad, they likewise found expression, especially in England, whose people had been roused for years regarding the future of their coal fields.

In October, 1902, Mr. Andrew Carnegie, one of our most distinguished iron-masters, was installed as Rector of the University of St. Andrews, Scotland. He delivered a very interesting address in which he stated that, if the rate of consumption of iron ore in the United States did not greatly increase, we would have a supply of first-class iron ore for only 60 or 70 years and of second-class for 30 years longer. Mr. Carnegie estimated our demonstrated store of unmined ore at one billion tons. The consumption, at that time, was between 25 and 30 millions of tons annually. All persons well informed upon mining matters would infer that the mining of a billion tons, now demonstrated, would reveal appreciably more; and while a billion tons divided by 25 gives a life of 40 years, 60 or 70 years was a not unreasonable figure. Yet this period is a relatively short one and the forecast justifies anxiety. Since Mr. Carnegie's address was delivered, the annual output of ore has doubled, and, unless relieved by other considerations, whatever apprehensions were justified then are twice as emphatic now.

In 1895, from three different spokesmen came prophecies similar to those of Mr. Carnegie. Sir Robert A. Hadfield, whose words regarding the iron and steel industry should carry as great weight as any man's, in a presidential address to the British Iron and Steel Institute [1] forecasts the call of the world's furnaces upon the mines at the outset of the new century, and upon the basis of known reserves also gave good ground for apprehension. In the same year, the late Professor Törnebohm, long the chief of the Swedish Geological Survey and with special experience in iron ores, made a report to the Parliament of Sweden, based on a visit to this country. [2] At this time, the Swedish government was actively sharing in the development of the great bodies of iron ore in Lapland, far within the Polar Circle. The importance of knowing the part which they might play in the world's iron industry of the future was great, and the determination of the

[1] Proceedings, 1905, I., 27, and especially 86-60.

[2] Reprinted in the Iron Age, Nov. 2, 1905.

limits of annual output was a matter in which the Swedish authorities felt a lively interest.

Professor Törnebohm credited the Mesabi range with half a billion tons; the other Lake Superior ranges, collectively, with as much more; and the Eastern brown hematites with sixty millions. This total of a little over a billion tons gave cause for anxiety, since the output in 1905 of American mines had risen beyond forty millions, and a life of twenty-five years was thus indicated. But, of course, a moment's reflection shows that the estimates are incomplete, since the Clinton ores of the East, and especially of Alabama, are omitted entirely.

In the same year, 1905, the late Professor N. S. Shaler sought to rouse his countrymen to an appreciation of the situation with regard to the mining industry, in a paper of a popular nature on "The Exhaustion of the World's Supply of Metals."[3] Professor Shaler, in general terms, considers the supply of ores of all sorts remaining to us as, roughly, twenty times the amount already mined. He thinks another century will exhaust the European supplies of iron ore. The best place for the iron industry is in the Mississippi Valley, and the ores tributary to it are passed in review without definite figures, except for Alabama, to whose Clinton red hematites a life of fifty years is assigned.

Other papers preceded, accompanied or followed the four specially cited and of these a list is given at the close of this contribution. They cannot all be mentioned now, and the ones briefly reviewed will suffice to show the apprehensive state of the public mind, here and elsewhere, from ten to fifteen years ago.

As a symptom of the widespread interest and as a natural step to prevent waste and to maintain as long as possible the material supports of industries, the conservation movement sprang up in this country. It has taken form in annual conventions and discussions, and has been influential in matters of legislation. Outside the American boundaries, similar steps have been taken. Reports of the Canadian Conservation Commission regularly reach us.

In connection with conservation in general, iron ore has been one of the chief subjects to be considered, and we are not

[3] International Quarterly, Vol. II, 230-247, 1905.

surprised to find our Swedish colleagues, as soon as they were assured at the International Geological Congress held in Mexico City, in 1906, that their invitation for the meeting of 1910 would be accepted, began to plan a great work on the "Iron Ore Resources of the World". Iron mining is one of the chief, if not the chief single industry in Sweden. The subject, therefore, possessed great local as well as international importance. The associated authors in all lands began to busy themselves at once with data and estimates of reserves. A year after the movement had been started by the Swedish committee and by its representative in this country, a special investigation of American iron ore reserves was also initiated under the United States Geological Survey, with Dr. C. W. Hayes in charge of the collection of data. The result of these endeavors led to the preparation of as complete estimates as were practically possible.[4] They will be mentioned and utilized later on.

Before we can actually undertake a discussion of the future, we must have clearly before us several matters of vital import. We must know the large features of production in the United States as a whole and in the more important individual districts. We need to briefly trace the progress of production during recent years. We need, further, to know what the general run of working percentages has been and to answer the questions: Is the yield per ton declining as the years pass, and are we content now to treat ores of lower grade than were our fathers? How do our ores compare in yield with those of foreign productive areas? We cannot overlook the vital bearing of our supply of coking coal—a factor in present iron metallurgy not inferior to ore supply itself. We must consider sources of ore outside the United States and yet so situated as to contribute to our furnaces. We must also consider present, or reasonably certain future, improvements in processes of smelting. No horoscope for the future can be cast without attaching due weight to all these factors.

The growth in the production of iron ore in the United States has been so great as to be the chief cause of anxiety for the future. The tabulation of a few figures, using a million long tons as the unit, will make the matter clear. Extended statistics

[4] C. W. Hayes, Bull. 394, U. S. G. S., 70-114, 1909.

are not necessary. I am extremely anxious that the great striking truths should not be lost in a maze of figures. The statistics are taken from the Mineral Resources of the U. S. Geological Survey. Detailed figures are not attainable for 1888 and earlier years, except in those in which a census was taken.

In the years before the Civil War, the production was small; but shortly after peace was restored the Lake Superior mines began to assume greater and greater importance, and later, Alabama developed its mining and smelting industry.

Statistics in Millions of Long Tons.

	United States Total	Lake Superior	Alabama	Other Eastern States	Western States
1860	2.8	......	......	......	......
1870	3.8	0.8	......	......	......
1875	4.0	0.8	......	......	......
1880	7.1	1.9	0.17	5.2	0.00
1882	8.7	2.9	......	......	0.05
1884	7.7	2.5	......	......	0.02
1886	10.0	3.5	......	......	......
1888	12.0	5.0	......	......	0.03
1890	16.0	8.98	1.90	4.96	0.19
1892	16.2	9.50	2.31	4.24	0.15
1894	11.8	7.60	1.49	2.70	0.29
1896	16.0	10.50	2.04	3.10	0.26
1898	19.4	13.8	2.40	2.84	0.37
1900	27.5	20.50	2.75	3.75	0.54
1902	35.5	27.05	3.57	4.30	0.65
1904	27.6	20.30	3.70	3.28	0.36
1906	47.7	37.80	3.99	4.91	0.80
1908	35.9	28.10	3.70	3.50	0.52
1910	56.8	46.30	4.80	4.80	0.80
1912	55.1	46.40	4.60	3.10	0.90
1914	42.0	32.91	......	......	......

GENERAL PROGRESS OF PRODUCTION.

By these figures a modest but steady growth in the production of iron ore is shown up to 1884. A marked increase then developed, which subsequent figures will show was chiefly due to the entrance of the Gogebic and Vermilion ranges. A rapid growth followed to 1890; and then production held steady, or, as in 1894, temporarily dropped back during panic times. Following 1896, the growth was very marked and was chiefly due to

the Mesabi range. Hard times checked it in 1904, in 1908, and again in 1914. No industry is more sympathetic with general business conditions than is the production of iron and steel.

The figures also show that the great increase in output is due to the growth of the industry in the Lake Superior region. Without the contributions from the Lake, the country as a whole would be back in the position which it occupied in 1886, with about ten million tons total production.

In general, if we look back to 1860 and take time by decades, we may say that today the production is twenty times what it was in 1860; fifteen times what it was in 1870; eight times that of 1880; three and one-half times that of 1890; and twice that of 1900. We cannot continue in the same ratio, but must ere long reach our zenith.

Production of the Lake Superior Ranges in Millions of Long Tons.

	Total U. S.	Marquette	Menominee	Gogebic	Vermilion	Mesabi	Cuyuna
1870....	3.8	.85	0.0	0.0	0.0	0.0	......
1875....	4.0	.88	0.0	0.0	0.0	0.0	......
1880....	7.1	1.38	0.52	0.0	0.0	0.0	......
1882....	8.7	1.83	1.14	0.0	0.0	0.0	......
1884....	7.7	1.56	0.89	0.001	0.06	0.0	......
1886....	10.0	1.63	0.88	0.75	0.30	0.0	......
1888....	12.0	1.92	1.19	1.43	0.51	0.0	......
1890....	16.0	2.86	2.27	2.91	0.89	0.0	......
1892....	16.2	2.84	2.40	3.06	1.23	0.03	......
1894....	11.8	1.93	1.25	1.52	1.05	1.91	......
1896....	16.0	2.42	1.76	2.10	1.20	3.08	......
1898....	19.4	2.99	2.27	2.55	1.12	4.83	......
1900....	27.5	3.94	3.68	3.10	1.67	8.16	......
1902....	35.5	3.73	4.42	3.68	2.06	13.08	......
1904....	27.6	2.46	2.87	2.13	1.05	11.67	......
1906....	47.7	4.07	4.96	3.48	1.79	23.56	......
1908....	35.9	3.31	2.90	3.24	.92	17.72	......
1910....	56.8	4.63	4.98	4.74	1.39	30.57	......
1912....	55.1	3.54	4.46	3.92	1.45	32.60	0.37
1914....	42.0	......	......	......	......	......	......

A brief survey of the figures relating to the individual Lake Superior ranges will justify the following conclusions: The Marquette, Menominee, Gogebic and Vermilion ranges show a steady, normal increase in output, which is not startling nor one to cause, under ordinary circumstances, undue apprehension. Some signs

of declining output are manifest in the case of the Vermilion. The vast increase in the output of iron ore is due to the Mesabi range, and from it in 1912 came nearly 60 per cent of the country's total. A marked decline in available supply from the Mesabi would bring about a greater falling off in ore supply than any possible increase in the other Lake Superior ranges, or than the present sources of supply from other mining districts, could make good. The Mesabi range is the key to the maintenance of the domestic supply at its present grade, and when it declines we must appeal to foreign sources to keep the iron and steel industry in its present position.

YIELD OF THE ORES.

Conditions vary greatly in different parts of the country; at different times; with different ores; and with the entrance of new sources of supply. It is a general truth that the richest ores are obtained in the early days of mining. As time passes and the industry becomes firmly established, lower and lower grades come within the range of profit. Alabama Clinton ores gave much higher percentages when mined wholly above the permanent water level than they do now, when pursued below it. For many decades, only lump ore, and much of that over 60 per cent iron, was produced by the magnetite mines of the eastern Adirondacks. Today the greater portion of the ore goes first through a magnetic concentrator before it is shipped. In earliest years on Lake Superior, hard, specular ore at 65 and above was sought. With improved facilities, the grade came down to and below 60, but the soft ores found slight sale. Now the soft, earthy ores are the principal objects of mining, and the average grade is well down in the fifties. Important shipments of ore with percentages below fifty have been placed on the steamships.

In the summer of 1875, Professor Albert H. Chester,[5] an experienced chemist, visited the Lake Superior region in the endeavor to secure average samples from the stock piles of the larger mines, all, of course, at that time in the Marquette range and shipping hard, specular ores. Four samples ranged from 61.01 to 66.83 and probably give a fair idea of the ore at that time sent away. Iron Mountain, Mo., ore ran 64.87; Lake Cham-

[5] Albert H. Chester, "On the Percentage of Iron in Certain Ores." Trans. Amer. Inst. Min. Eng., IV, 219, 1875.

plain magnetites, 56.01 to 62.68; Clinton, N. Y., fossil ore, 44.57, but yielded 43 in the furnace.

In September, 1890, Geo. W. Goetz [6] published a tabulated series of analyses from the four older Lake Superior ranges, which, when averaged, afford the following values. To give a correct average, the analysis of each mine's ore ought to be weighted with the output, and as the data for this calculation are not available, we must be content with the general significance of the results. On the whole, they supply us with trustworthy values.

Range	No. of Analyses	Maximum	Minimum	Average
Marquette	36	69.77	53.02	62.33
Menominee	23	65.20	52.18	60.00
Gogebic	21	65.45	54.95	62.09
Vermilion	8	67.54	60.20	64.50

These figures represent the good old times when specular ore was almost the only one produced and before the soft ores began to be a serious factor. They are, however, significant, in that customary working percentages, such as these, very probably were not without their influence in the estimates of the life of the ranges, as set forth by several of the writers whose opinions were cited in the introduction to this address.

Raphael Pumpelly, in connection with the summaries of the Tenth Census,[7] estimated on the best and most comprehensive data which we have ever had, the general average of iron ores for the United States at 51.22 per cent iron. The maximum average percentage among the States was that of Missouri, 60.01 (but Michigan had 59.57). The minimum was West Virginia, 37.92. Pennsylvania, the largest producer of ore in that year, gave 45.28. On the basis of ore production and pig iron production, allowance being made for mill cinder, foreign ores, etc., John Birkinbine estimated for the Eleventh Census [8] an average of 51.27 for the country at large. An appreciable error crept in, however, in assuming pig iron to be entirely iron, whereas it is only about 95% or less metallic iron. We can hardly compare this figure with the one given by Professor Pum-

[6] Geo. W. Goetz, "Analyses of Lake Superior Iron Ores," Idem. XIX, 59, 1890.

[7] Tenth Census, Vol. XV, 19, for the year 1879.

[8] Volume on Mineral Industries, p. 10.

pelly which was based on actual analyses of samples. If we credit the 7,000,000 tons of pig iron, as used by Mr. Birkinbine, with 95% iron, the average is 48.71, which indicates an appreciable falling off in yield in ten years.

General estimates of average percentages which will be trustworthy are difficult to carry out on the basis of annual statistics of tons of ore and tons of pig iron. Foreign ores contribute to an appreciable degree, and their yield can only be estimated. Stocks of mined ore, stored at furnaces or mines at the end of a year, are naturally credited to that year, but they are not turned into pig iron until the following twelvemonth. Mill-cinder is also a contributor of iron to the extent of a small percentage of the total. The data for all these corrections are not available for a long period of years, and, therefore, all could not be introduced in the following estimates. The importations could, however, be deducted, and to them an average of 58 percent iron has been arbitrarily assigned. The results obtained are so variable that their significance is rather one of degree than of actual individual accuracy. The statistics are chiefly taken from the Mineral Resources for 1910, p. 76. Long tons are used.

	United States Iron Ore in thousands of long tons	Imported Ore in thousands of long tons	Pig Iron in thousands of long tons	95% Pig Iron, Metallic Iron, in thousands of long tons	Iron in Imported Ores at 58% in thousands of long tons	Net Iron	Average percent of Ore	Estimated by E. C. Eckel, "Iron Ores," p. 358, 1914
1870	3,832	....	1,678	1,594	....	1,594	41.6	....
1875	4,018	56.6	2,040	1,938	33	1,903	47.3	....
1880	7,120	493	3,802	3,612	286	3,326	46.7	....
1885	7,600	391	4,077	3,874	227	3,647	48.0	....
1890	16,302*	1,247	9,203	8,653	723	7,930	48.6	56.50
1895	17,203*	524	9,446	8,974	304	8,670	50.4	54.95
1900	26,722*	898	13,789	13,100	521	12,579	47.1	51.55
1905	43,433*	846	22,992	21,842	491	21,351	49.1	53.19
1910	55,246*	2,591	27,304	25,939	1,501	24,439	44.2	49.42
1912	58,031*	2,104	29,727	28,241	1,220	27,021	46.5	51.46

* These totals are the apparent iron ore consumptions as given in the Mineral Resources, U. S. Geol. Survey, for 1912, p. 162. They differ from the totals of production in the previous tables because corrected for unsmelted stocks, exports and zinc residuum. No correction is made for mill cinder.

The variations shown above are so pronounced as to cast some doubt upon the accuracy of the individual percentages, but we may have some confidence in the general tendencies shown. We cannot but be impressed with the apparent practice of the mining companies of using lower grade ore in good times, as shown by high production, and saving higher grade ores for bad years. So far as recent years are concerned, we can only say that the general grade has declined, although it does not appear to be as low as it was in 1870, when the brown ores of the East were so large a factor in production. It must be today well below 50 per cent.

In the last column, and for the years beginning with 1890, are given calculations of average yield, prepared by E. C. Eckel in his valuable manual on ''Iron Ores'', published in 1914. The same figures for apparent iron ore consumption have been used as in the calculations given in the first column of the present table; that is, the total annual production has been increased by imports and by zinc residuum (i. e., used for spiegeleisen by the New Jersey Zinc Company), and diminished by exports and by stocks on hand at the close of the year. The zinc residuum is only 0.2 to 0.4 per cent of the total and makes little difference. But a decided difference arises in calculating the yield of American ores if one assumes that pig iron is pure iron, and lets the much richer importations of foreign ores enter into the calculation. These last two elements in the problem explain the wide divergence in percentages of from 4 to nearly 8 per cent between the average values given in this paper and those quoted by Mr. Eckel. Both calculations depart from the truth in so far as mill cinder, blue billy, scrap iron, etc., enter into the problem, since no account has been made of them. Of course, there is also a slight loss of iron in blast-furnace cinder.

The great importance of the decline in yield is the vastly increased amount of reserves which are thereby brought within the range of mining. As the average may still further decline until it reaches, say, 35 per cent, the reserves, as figures to be given later will show, become enormous. Thirty-five per cent, however, is by no means an unreasonable figure for the general yield of the Jurassic ores in the Lorraine and Luxembourg districts, which so largely supply Belgian, French and German

furnaces. The same statement will apply to the Cleveland district in England. The great reserves of thirty-five per cent ore in the Lake Superior district are, however, highly siliceous, whereas the Jurassic ores are basic. In Silesia, in southeastern Germany, even lower percentages are not esteemed beyond the possibilities. Thirty-five per cent is therefore a not unreasonable figure to consider, when a long look ahead is taken. On the other hand, in comparing the yield of the ores in different lands, a distinction should be made between exporting and smelting countries. Exporting countries necessarily must furnish high grade ore, so as to meet freight charges incident to long transportation.

ESTIMATES OF RESERVES.

Since 1905, several estimates of reserves have been made, of which condensed summaries may be cited.[9] The amounts are in millions of long tons.

1905	Törnebohm	
	Lake Superior	1,000
	Alabama	60
	Elsewhere	40
		1,100
1907	E. C. Eckel	
	Lake Superior	1,500-2,000
	Alabama Red Ore	1,000
	" Brown Ore	75
	Georgia Red Ore	200
	" Brown Ore	125
	Tennessee Red Ore	600
	" Brown Ore	225
	Virginia Red Ore	50
	" Brown Ore	300
		4,075-4,575

Southern reserves for the remote future were estimated at 10,000 million tons.

1909	Butler-Birkinbine	
	Lake Superior	1,618
	Southern States	1,814.9
	New York	750
	New Jersey	135
	Pennsylvania	45
	Rocky Mountain Region	100
		4,462.9

[9] The figures as given for Törnebohm, Eckel and Butler-Birkinbine are cited from E. C. Eckel, "Iron Ores," 341-351, 1914.

1911	Minn.-Michigan Tax Commission, J. R. Finlay, Engineer	
	Minnesota and Michigan	1,584
1912	E. C. Eckel	
	Lake Superior	2,000-2,500
	Northeastern	300- 600
	Western	300- 700
	Birmingham	1,500-2,000
	Texas	600-1,000
	Other Southern States	500- 750
		5,200-7,550

The most complete of all the estimates is that of Dr. C. W. Hayes in Bulletin 394 of the U. S. Geological Survey, 1909. The estimates are divided into two classes of ores; first, those available under present conditions; and second, those which come within reasonable possibilities of utilization for the future. The statistics are given in long tons in millions and decimals of a million.

Available Ores.

Districts	Magnetite	Sp. and Red Hem.	Clinton Ore	Brown Ore	Carbonate Ore	Total
Northeastern	160.0	2.0	35.0	11.0	......	208.0
Southeastern	12.5	8.0	463.5	54.4	......	538.4
Lake Superior	....	3,500.0	10.0	....	......	3,510.0
Minas Valley	....	15.0	....	300.0	......	315.0
Rocky Mts.	51.5	4.3	....	2.0	......	57.8
Pacific Slope	68.9	....	....	....	......	68.9
Total	292.9	3,529.3	508.5	367.4	......	4,698.1

Titaniferous magnetite considered available by Dr. Hayes	90.0
	4,788.1

Not Available Ores.

Districts	Magnetite	Sp. and Red Hem.	Clinton Ore	Brown Ore	Carbonate Ore	Total
Northeastern	211.5	2.0	620.0	13.5	248	1,095.0
Southeastern	23.0	53.0	970.5	168.0	62	1,276.5
Lake Superior	4,525.0	67,475.0	30.0	....	......	72,030.0
Miss. Valley	....	10.0	....	560.0	......	570.0
Rocky Mts.	116.9	2.1	....	1.6	......	120.6
Pacific Slope	13.8	10.0	....	0.1	......	23.9
Total	4,890.2	67,552.1	1,620.5	743.2	310	75,116.0

In the last group of ores, I have included Dr. Hayes' estimates of titaniferous magnetites, without separate classification.

The estimates for the Eleventh International Geological Congress were grouped in a somewhat different manner, as follows:

	Available	Probable Addition
Archean Magnetites—		
Lump Ores	20.0	30.0
Concentrates	40.0	10.0
Adirondack Red Hematites	2.0	2.0
Pennsylvania Soft Magnetites	40.0	....
Cambro-Ordovician Brown Hematites	65.0	181.0
Mesozoic and Tertiary Brown Hematites	10.0	15.0
Clinton Red Hematites	505.3	1,368.0
Alabama Gray and Red Hematites	27.5	27.5
Carbonate Ores	....	308.0
Lake Superior Hematites	3,500.0	72,000.0
Miss. Valley Specular and Red Hem.	15.0	5.0
" " Palaeozoic Brown Hem.	30.0	45.0
" " Tertiary Brown Hem.	260.0	520.0
Cordilleran Magnetites and Hematites	63.8	55.0
	4,578.6	74,566.5
Titaniferous Ores	90.0	128.5

As shown earlier, the annual production in recent years has totaled between 50 and 60 millions of tons. Let us assume that it will be 60 millions in the near future. Dr. Hayes' estimates indicated practically 4800 millions of tons of available reserves or eighty years' supply. The estimates for the International Geological Congress of 1910 are not appreciably different. By just so much as the annual production exceeds the amount of 60 millions, will the time be shortened, except in so far as further exploration opens up new reserves. In mining enterprises in general, however, if the management of a company felt that it had eighty years fairly well assured, it would congratulate its stockholders on the outlook. This attitude of mind would be justified by the common experience in mining the ores of such a widely distributed metal as iron, that new reserves open up in old or new properties as old supplies are exhausted.

On the other hand, if we anticipate the general decline in the yield of ores, so that lower and lower grade reserves may be brought in; and if we assume that more tons of ore will be required to furnish the usual output of pig iron, such that the annual output of ore may reach 100 millions; then from the

probable addition of reserves, given in the second column of estimates, we forecast from practically 75,000 million tons; a life of 750 years. That iron could be produced in these amounts and for this period of time, there can be no doubt, if we omit consideration of cost and if we only consider possible ores down to 35 per cent. Iron-bearing rocks of still lower percentages are so abundant as to be inexhaustible. No one need feel anxiety about the physical possibility of producing iron up to the conceivable life of the race on the planet.

In earlier pages, the point was emphasized that the crux of the present situation lies in the Mesabi range of Minnesota. Of the 55.1 million tons produced in 1912, 32.6 millions came from it. The chief point of immediate interest, therefore, is concerned with the life of the Mesabi. Its decline means great rearrangements in the present situation in the iron industry. The most recent estimates are those of C. R. Van Hise, C. K. Leith and W. J. Mead, in cooperation, as given in Monograph 52 of the U. S. Geological Survey, 1911. Fifty per cent of iron in the dried ore is assumed as the minimum average yield at the time the estimates were made; 1600 millions of tons were then credited to the Mesabi (p. 489). The output for 1910, for this range, was 30.57 millions, indicating a life of a little over 50 years. At the production of 32.6 millions for 1912, a life of almost exactly 50 years is shown. If, on the other hand, a minimum percentage of 35 in iron is considered, the same authors assign to the Mesabi range reserves of 30,000 million tons (p. 492), which would give us 300 years of life, even at 100 million tons annual output.

The authors of Monograph 52 also discuss the reserves of the entire Lake Superior region. The reserves of 50 per cent ore, in the other ranges than the Mesabi, are less than one sixth its amount, and their combined output about two fifths its total. Their estimated life is thus much shorter. The time period lies between 20 and 25 years. When, however, we consider a minimum yield of 35 per cent, their combined reserves are greater than those of the Mesabi, and are estimated at 37,630 millions of tons. If we credit them with two to three times their present annual output, a life of fully a thousand years is shown.

Thus one can attack the problem from various points of

view and with varying assumptions; but the conclusion is inevitable that the output of ore from the Lake Superior region cannot be kept up at present production and with a minimum yield of fifty per cent for as much as fifty years, unless unanticipated new discoveries of rich ore are made. With diminishing yield, however, and with the tenor still at percentages above 35, the shipments of iron ore, even in increasing amounts, can be maintained for centuries.

Let us turn next to Alabama and its closely related states, Georgia and Tennessee; since, together, they constitute the second center of ore production. The great reserves lie in the Clinton ores, which are well stratified and which have been and will be explored by bore holes. The reserves are much increased by the brown ores of the region and of northwestern Alabama, and by the probable development of much older gray and red hematites in eastern Alabama; but attention will be alone directed at this point to the Clinton ores. The latter are so well stratified and persistent and are now proved by such extensive exploration, that with much confidence we may credit them, at least in the Birmingham region, with 36 to 37 per cent iron, and may consider the estimates of reserves as unusually trustworthy. Dr. C. W. Hayes, on the basis of the careful field work of C. F. Burchard,[10] estimated them at the following amounts in millions of tons:

	Available	Not Available
Tennessee, Georgia and Northeast Alabama	86.5	440
Birmingham District, Ala.	358.5	438
Total	445.0	878

Mr. E. C. Eckel had previously credited the Birmingham district with 1000 million tons, a number not unduly above the sum of the two figures for Birmingham given above. The officers of the Tennessee Coal and Iron Company considered, in 1909, in round numbers, 500 million tons as reliably assured.

The combined output of these three states in Clinton ore was practically 4 millions of tons in 1912, indicating at this rate 111 years' life assured, and over 200 years' additional life as probable. In these estimates we do not assume an essential

[10] Bulletin No. 394, U. S. G. Survey, pp. 88-89, 1909; No. 400, pp. 129-133, 1910.

falling-off in the yield of the ores below percentages actively mined today.

Were we to take up the figures for the other portions of the country, very similar results would be reached. But, as their contributions are proportionately smaller, the effects of rearrangements are less serious. Obviously, in a general way, viewing the country at large, and allowing for reasonable decline in yield, the ore supply is good for several centuries.

FOREIGN SOURCES OF SUPPLY.

The yield in the furnace is certain to be maintained, in an important manner, by importations of rich ores from abroad. These contributions are already a serious factor, since they amounted to 2.1 million tons in 1912, and had reached 2.5 millions in 1910, ranging between 3.5 and 4.6 per cent of the total.

Cuba: The most accessible and the heaviest contributor of ore is Cuba. The mines in the vicinity of Santiago, on the southeastern coast, have been shipping for twenty years amounts which annually range below and above a half million tons of magnetite, with some hematite mechanically intergrown. The ores now run from 55 to 60 per cent in iron and are of bessemer grade. For some years additional, these contributions will continue. The great and enduring reserves, however, are on the northeastern coast or near it. Extensive areas of serpentine have weathered in the tropical climate so as to afford a heavy mantle of alteration products, which, when freed of absorbed water yield 48 per cent iron, with about 1 per cent nickel and 1 to 2 per cent chromium. When freed of additional combined water in calcining furnaces, the ore reaches 56 per cent iron. The Mayari tract, already actively mined, can yield 600 million tons of excellent nickel-bearing bessemer ore. The undeveloped Moa and San Felipe (or Cubitas) districts can swell the reserves to 2000 million tons. Thus, as the output of the mines in the United States falls lower and lower below present percentages, more and more can the grade be kept at or near the above values by Cuban contributions to furnaces near the Atlantic seaboard. The supply of Cuban ores is sufficient to last several centuries, at any reasonable consumption

of conceivable importations. They are very conveniently situated for low costs of mining and shipping.

Sweden: In recent years, the second contributor to American furnaces has been Sweden. The supplies have come from the great magnetite body at Kiruna, in Swedish Lapland. The ore reaches the sea at Narvik, in Norway, a port open all the year round, and distant from the mines 100 miles by rail. A generally high phosphorus ore is now mined, with a small proportion of rich bessemer grade. The output is sorted into different grades, possessing from 59 to 69 per cent iron, with perhaps a general average of 65. Importations in 1912 into this country were practically 334,000 tons. The output of the mines is carefully regulated by the Swedish government with the purpose of conserving the supply for a long life. The United States cannot anticipate more than a moderate contribution from this source.

Norway: In Norway, not far from the sea and adapted to magnetic concentration, there are additional deposits which are possibilities for the future. One enterprise is already active on the extreme northeastern frontier of Norway, east of the North Cape. The European furnaces have, however, absorbed the output, hitherto.

Newfoundland: The third source of importations, in recent years, has been Newfoundland. The shipments come from the red hematite mines on Bell Island in Conception Bay. The ores are beds of red hematite in Cambrian and Ordovician strata and are strongly reminiscent of the Clinton ores. They supply a non-bessemer ore of 50 per cent, or slightly less, in iron, and in their best years have exported over 200,000 tons to the United States. The reserves which run beneath the sea are estimated by J. P. Howley at over 3000 millions of tons. The ores are generally called the Wabana. With a sea voyage of 1100 to 1500 miles, they can reach our principal ports of entry. Their chief markets, however, are the iron and steel centers of Nova Scotia.

Chile: The Panama Canal has made accessible one great deposit of iron ore on the west coast of Chile, called the Tofo. Tofo is 30 miles north of Coquimbo. The ores are only three or four miles from the sea. The Bethlehem Steel Company is

making extensive preparations for shipments on a large scale in the immediate future. Published descriptions mention reserves of 100 million tons of ore ranging above and somewhat below 60 per cent and prevailingly of bessemer grade. A possible annual output of 1.5 to 2 millions of tons is expected. (Iron Age, May 11, 1914.) Other deposits along the west coast of South America have been reported in an incomplete way, but are not yet sufficiently developed to seriously enter into our forecasts.

Brazil: For some years past, reports have been current of very large, rich, low-phosphorus deposits of specular hematite in the State of Minas Geraes, Brazil. They constitute beds in metamorphic sediments of Precambrian age, and appear some three hundred and seventy-five miles from the sea-coast. Deposits of hard, specular hematite and loose blocks on the surface are available in enormous quantity. The first estimates, for the Eleventh International Geological Congress, by Orville A. Derby, the able State Geologist of Brazil, gave 2000 million tons. Since then, the observations of Leith and Harder indicate more than three times this amount. Vast quantities run between 65 and 70 per cent in iron and are well within bessemer limits. The chief handicap lies in the long railway haul to the sea. While railways tap the district, both from Rio Janeiro and Victoria (the latter the probable port of future shipments), the present roadbeds are not adapted to the hard wear and tear of a heavy iron ore traffic, and must be rebuilt.[11] Once on shipboard, the distance to Atlantic ports is about 4000 miles.

Europe and Africa: The United States also import appreciable amounts of ore from Spanish, Algerian and Grecian ports. Spain is the chief contributor; approximately 440,000 tons reaching Atlantic ports in 1910. To some extent, therefore, declining American percentages may be raised by future shipments from these sources; yet as time passes, British and Continental needs will be even more pressing than American and will call more insistently for supplies from European and Northern African mines.

The possibilities of importation and sale turn, however, upon market conditions. Through the kindness of Mr. Charles

[11] The latest account is by E. C. Harder, "The Iron Industry of Brazil," Transactions of the American Institute of Mining Engineers.

F. Rand, President of the Spanish-American Iron Company, the following figures have been supplied the writer. They summarize market conditions and ocean freights as they have prevailed in recent years:

Ocean freight from Cuba is ninety-five cents a ton; from Wabana, Newfoundland, seventy cents; from Brazil, two dollars twelve and one-half cents (i. e., eight shillings, sixpence); from Sweden, one dollar and a half; from Spain, one dollar thirty-seven and a half cents; from North Africa, one dollar, twenty-five cents; from Chile, three dollars. When the ore reaches American ports, it brings as a general rule seven cents a unit, although specially rich and pure varieties may command eight cents. From these data, in a general way, one can see the market conditions which must be met by an exporter of ore from any one of the countries which are the chief contributors to American furnaces. Ocean freights, for some time to come, certainly will not be less than in recent years, even when sea-going bottoms can be secured.

THE SUPPLY OF COKE.

So long as iron ore is turned into pig iron as the first step toward steel, as in our present-day practice, coke will be no less vital to the industry than ore itself. The relatively great height of a modern stack and the heavy burden of charge which rests upon the still burning fuel demand strong and resistant coke. Not every coke will answer. From an address by Mr. J. E. Johnson before the Mining and Metallurgical Society of America, Jan. 12, 1915, the following figures are taken: From 52 per cent iron ore a ton of pig iron may be made with one ton of coke. These conditions are approximately those of Lake Superior ores today. From a 38 per cent ore, a ton of pig requires 1¾ tons of coke, conditions approximately those of Alabama. Should we ever use 25 per cent ore, 2¾ tons of coke will be necessary to the ton of pig. Whatever may be said, therefore, regarding the coke supply today will apply with increasing force as the years pass and the yield of ores declines. Anthracite coal has been, to a certain extent, used in the iron furnaces, but its desirability and increasing price for household fuel and for steam purposes in our eastern

cities make it a factor in future iron metallurgy of diminishing importance. Open-burning bituminous coal has been used raw to some extent, but is not now a serious factor.

The following table summarizes the bituminous coal reserves as calculated by M. R. Campbell of the U. S. Geological Survey and as given in the Mineral Resources of the United States for 1910, p. 28. Only eastern coke-producing states are selected because the present effect of Rocky Mountain States upon the total result is not great. The influence which they can exercise upon the future is small or remote. The same is true of the Pacific Coast and its possible future industry in iron and steel. In the table the total bituminous coal reserves have been reduced by an arbitrary fraction, which is assumed to represent the portion of coking grade suitable to blast-furnace use. Much difference of opinion might arise over this reduction. Its importance turns, however, upon the ultimate result; that is, if the supply of coke proves to be a less serious matter than the supply of ore, these fractions might vary widely and yet not destroy the reliability of the final result. In the further calculations, I assume that two thirds of the coal can be ultimately mined, one third being left in pillars. In passing from coal to coke, I use the same percentages of yield for each of the states as are given in the Mineral Resources of the United States Geological Survey for 1912, Part II, p. 251. The estimates are, moreover, within the probable reserves, in this additional respect, that no account is taken of Illinois, although its weak coking coals, when mixed with others in by-product ovens, give suitable fuel for blast-furnace use.

Reserves of Bituminous Coal of Coking Grade in Millions of Long Tons.

	Total Bitum.	Fraction for Coking		Two-thirds Mined	Per Cent	Coke
Pennsylvania	109,174	1/4 =	27,300	18,200	66.5	12,100
Ohio	85,156	1/10 =	8,515	5,676	69.2	3,927
Maryland	7,802	1/4 =	1,950	1,300	65.8	855
Virginia	22,391	1/2 =	7,464	4,976	62.2	3,095
W. Virginia	149,120	1/4 =	37,350	24,900	60.7	15,114
Eastern Kentucky	67,687	1/5 =	6,768	4,512	62.4	2,815
Western Kentucky	36,104	1/10 =	3,610	2,406	62.4	1,501
Tennessee	25,509	1/4 =	6,377	4,251	54	2,295
Georgia	920	1/2 =	460	306	50	153
Alabama	68,594	1/3 =	20,865	13,910	64.9	9,027
	572,457		120,659	80,437		50,882

The production of pig iron, by states in 1912—the maximum year as yet—is given in the statistics in the next table in millions of long tons. The figures are taken from the Mineral Resources for 1912 of the U. S. Geological Survey. If we assume that the coke consumption per ton of pig is one ton in those states where Lake Superior ores, or others equally rich, are used; one and three-quarters tons in Alabama; and one and one-half tons in West Virginia and Virginia—we can make a rough estimate of the coke consumption for pig iron manufacture in a maximum year.

1912 Pig Iron Production in Millions of Long Tons, by States.

	Pig Iron	Coke Consumed
Pennsylvania	12.55	12.55
Ohio	6.80	6.80
Illinois	2.89	2.89
New York	1.94	1.94
Alabama	1.86	3.25
Indiana, Michigan	1.77	1.77
Missouri, Colorado, California	0.40	omitted
Tennessee	0.34	0.60
Wisconsin, Minnesota	0.30	0.30
West Virginia	0.27	0.40
Virginia	0.26	0.39
Maryland	0.22	0.22
Others	0.12	0.15
	29.72	31.26

We have thus an apparent available coke supply of 50,882 million tons, and a consumption for blast-furnace purposes, in our heaviest year of production, of 31.26 millions. There are thus over sixteen hundred years' supply at this rate. In Pennsylvania, on the assumed ratio of coking coal, there is about one thousand years' supply. These time periods are so great that despite possible errors in assumptions; despite increasing coke consumption with lowering of grade of ore; and despite increasing output of pig iron, we seem justified in concluding that the fuel supply is rather more abundant than the ore supply. The reserves of bituminous coal in 1912 were placed by the volume on Mineral Resources for that year at 1,651,057 millions of short tons of which two thirds or 1,100,705 short tons

could be mined. With an annual production, as in 1912, of 450 million tons, a life of nearly twenty-five hundred years would be indicated. Apparently coal for general fuel will last longer than coal for coke.

THE INCREASING STOCK OF SCRAP IRON.

Much of the iron or steel, once it is used, is lost by oxidation, wear and tear, or by being thrown away. A goodly proportion is, however, returned to furnaces and worked over. For this purpose, in America, the electric furnace has proved of special advantage, as the writer learns from Professor J. W. Richards. With growth of production and with increasing attention to the prevention of waste, now so generally manifested throughout the country, the return of old iron and steel for re-treatment is likely to ease somewhat the strain on the mines.

IMPROVEMENT IN PROCESSES.

Electrical processes of smelting, in regions of great water powers and low cost for current, have excited hopes of saving fuel. The fuel in the blast-furnace accomplishes two purposes —the production of a high temperature and the reduction of the iron oxide to the metallic state. The electric furnace could serve to replace the former portion, but carbon for the reduction of the iron oxide would always be necessary. Some heat, of course, would be developed in the reaction itself, which practically implies the combustion of the carbon. If we assume a practicable electric furnace, comparable so far as the installation is concerned with a blast-furnace, we have to balance against each other the cost of heat from combustion of coke and from electric current. Thus far coke has proved more economical, although it is conceivable that countries like Sweden and Norway, with abundant water power and ores, but without coals, might develop an electric smelting industry. Charcoal would probably then furnish the reducing agent. For some time to come, we can see little chance for electric smelting in eastern North America.

Improvements are then reduced to those possible for the blast-furnace itself. We are reminded of the great economies

introduced by the chilling and separation of the moisture in the air to be used in the blast. A great debt is due Mr. James Gayley for this invention, which steadies the running of the furnace and keeps conditions uniform. We recall the use of the spent blast in internal combustion engines, and the economical generation of power in this way instead of through the ordinary medium of steam. The power is then available for all manner of applications around a works, and lowers costs. We note the recent and very encouraging experimental run of some months at the Port Henry, N. Y., furnace, with large proportion of titaniferous magnetite in the charge. The reports of Mr. J. E. Bachman,[12] in charge of the furnace, do much to remove the stigma from this variety of ore, and to make available large reserves now looked upon with suspicion. By just so much as these neglected ores come into use, the life of the non-titaniferous varieties will be prolonged. Dr. C. W. Hayes[13] estimated the titaniferous ores in 1909 at 90 million tons available and 128.5 million tons as not at present available. Dr. J. T. Singewald[14] has concluded that in some of the areas used in the calculations of Dr. Hayes, the ores are too low for probable use. These ores have not been very generally explored as yet, because of their bad reputation, but the amount is quite certainly large.

A remote possibility for improvements in the blast-furnace, but one worthy of careful consideration, was suggested by Mr. J. E. Johnson in the address at the annual meeting of the Mining and Metallurgical Society of America, January 12, 1915, which has been already cited. The air passing through the furnace is, by volume, nearly four-fifths inert nitrogen, which contributes nothing to the reactions and is a serious absorber of heat. Were it possible to relatively increase the proportion of oxygen, loss of heat might be avoided and fuel consumption reduced. Mr. Johnson called attention to the production of greatly enriched proportions of oxygen by the expansion of liquid air under suitable control, as now used in practicable

[12] The Iron Age, Oct. 22, 1914, p. 936; Dec. 24, 1914, p. 1470. A complete report is in press in the publications of the Iron and Steel Institute.

[13] C. W. Hayes, Bulletin 394, U. S. Geological Survey, p. 102, 1909.

[14] J. T. Singewald, Bulletin 64, Bureau of Mines, p. 38, 1913.

processes for obtaining oxygen on the one hand and nitrogen on the other. Were it possible with the low-cost power, to be developed by the products of the blast-furnace, to manufacture liquid air, or to produce in the same general way a strongly enriched oxygenated air for the intake, the volume of atmospheric gases would be greatly reduced and the heat economies would ensue. The contrast presented by employing the coldest substance known, as a means of facilitating one of the hottest reactions of technical practice, is so novel as to arrest attention. Costs, however, should it ever become practicable, place it in the remote future.

A more immediately practicable economy, involving the saving of waste, is the use of blast-furnace cinder for the manufacture of cement. By just so much as this ordinarily rejected product can be made a source of financial return, costs will be reduced. While we may not realize the whimsical ideal presented by Mr. Johnson in the above address, when he pictured the furnace of the future as yielding pig iron at the tap and cement at the cinder notch; yet we may think of slag utilization as helping to usher in the next age of the world, the one which is rapidly displacing the present Steel Age—the one which we all recognize as the inevitable Age of Cement.

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Paper No. 107

THE LIFE OF IRON AND STEEL STRUCTURES.

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This paper deals chiefly with open-hearth mild steel in civil engineering construction in the United States, and this material is understood to be referred to when the contrary is not explicitly stated. The principal attention is directed to steel because its present use is vastly greater than that of wrought iron, and because cast iron is not extensively used in the United States for structural purposes, except for columns and ornamental construction and some special purposes such as tunnel and shaft linings, pipes, pedestals, etc.

The enormous amount of structural steel produced in the United States (more than 31,000,000 tons in 1913), its great strength and uniformity, and the great facility of fabrication and rapidity of erection have made it an almost universal element in almost every kind of permanent or temporary important construction.

It is adopted by such a multitude of competent and incompetent designers for such widely different purposes, varying or uncertain conditions and requirements, that an infinite amount of research, analysis and classification would be necessary to formulate any reliable conclusions as to its ultimate life, even if its history to date were long enough to justify them, which it emphatically is not.

Such an investigation is worthy of, and demands the ability and resources of a commission of national experts, investigating and studying for a period of years. The limited time

and facilities available for this paper do not permit either elaborate or authoritative treatment of the subject, therefore the effort has been made to outline some of the essentials for consideration, indicate in some cases their generally accepted import, and present the data and opinions furnished by eminent and experienced specialists.

The tremendous importance of the life of iron and steel structures may be appreciated by a consideration of the total annual production throughout the world, which, in 1913, reached a maximum of 78,000,000 tons of pig iron, of a value of approximately $11,000,000,000. The United States alone produced 31,300,874 tons, and a still larger quantity of steel, owing to the fact that much of the latter was manufactured from scrap. If we assume, with Mr. James Aston, that there is a loss of 1,000,000 tons annually due to corrosion, it would involve a direct money loss of not less than $20,000,000, without regarding the depreciation of the remaining metal, which would be much greater. It might perhaps be more conservative to assume that the structural steel product amounts to 23,000,000 tons, or about 30 percent of the pig-iron product, and that it suffers a total average depreciation due to all causes, but chiefly from corrosion, electrolysis and similar causes, of 3 percent per year. Assuming that the erected cost of structures is at least $40 per ton, this would amount to about $27,600,000, an estimate which is probably within the truth.

The life of a steel structure depends upon three fundamentals: the character of the structure, the service it performs, and the condition under which it serves.

Character includes type, general and detail design, fabrication and erection.

Service Performed includes all the uses to which it is put, directly or indirectly, properly or improperly, and the stresses resisted.

Conditions of Service include the adjustment of the structure, its maintenance, protection, care, minor renewals, repairs, and all atmospheric, chemical, mechanical, dynamic, electrical, temperature and other physical influences other than service elements, to which it is subjected.

With perfected character, proper service, and suitable, uniform conditions, the life of most steel structures should be long; and of some, practically unlimited. For many types of structures, economic reasons frequently do not require or even justify extreme length of life, since they may become obsolete or service requirements may change so radically that their value is destroyed or greatly impaired long before they reach the limits of potential life. Or it may be proved economical to build a structure designed for a limited term of service and renew it after a certain period, rather than incur the heavier fixed charges of a more durable structure.

These considerations have been more or less recognized in the design of steel structures; and this fact, together with faults in many of the structures, improper service and unsuitable conditions—factors which in many cases are concealed—makes it impossible to attain great accuracy in deductions from a large proportion of existing structures.

Investigation is further greatly handicapped by the almost universal absence of accurate, detailed records of service and conditions, even when complete data sometimes, although seldom, are available of their original character. Finally, the very short time, less than forty years, since structural steel has come into extensive use is wholly inadequate for the demonstration of anything like the maximum life or useful service of many kinds of structures.

This paper gives records of a few structures which have had completed lives; describes the effects of some important factors in the life of structures; makes some deductions on reasonable and suitable life and the requisites of its attainment; enumerates the principal factors which should be investigated and considered in a comprehensive study of the life of steel structures, and presents a number of opinions of eminent authorities.

The opinions, while not intended to be considered as scientifically or mathematically established, are of great value as indicating the trend of advanced practice and careful design, and represent the net result of cumulative experience, the conscious or unconscious synthesis of study and observation, and the crystallization of practical and theoretical considerations

to positive, even if informal, conclusions. These are very useful in that they indicate the direction and manner in which the greatest useful life of certain structures can be advantageously and practically attained, and what its duration may reasonably be, under normal conditions and with present facilities for design, construction and maintenance. This is of far more value than a thesis on the theoretical life under assumed or ideal conditions.

The most important types of steel structures are bridges, buildings, boats, foundations, storage structures, aqueducts, subways, tunnels, ships and barges, contractors' plant and miscellaneous. The average actual lives of each of these types of structures vary greatly, as do the possible and theoretical maximum life. Reasonable estimates may, however, be formed, where sufficient data are available, of the probable life under ordinary conditions, and of the life that should be secured by proper design, construction and care. For efficient consideration of the problem, these types of structures should be subdivided into classes, and data for each accurately observed, tested and recorded, thus forming the basis for a complete and authoritative treatment of the subject, which is now impossible.

Bridges should be separated into railroad, electric car, and highway bridges; and their principal parts should be studied rather than the structure as a whole, thus separating riveted trusses, pin-connected trusses, girders, floors, towers, piers, cables, anchorages, expansion bearings, and the operating mechanism of movable bridges.

Buildings include tall office buildings, auditoriums, churches, hotels, tenements, mills, power and electric plants, industrial plants, markets, lighthouses, sheds, warehouses, stations, pavilions, theaters, some residences, grandstands, stadiums, and a multitude of light, portable structures like contractors' offices, garages, etc. The members comprised in them include columns, beams, girders and trusses for floors, walls and roofs, wind bracing, foundation grillages, floorplates, sheathing and roofing, stairs, furring, metallic lath, doors, shutters, window sash, bulkheads, screens, railings, and ornamental work, shingles and ceilings.

Boats include everything from battleships, submarines, ocean liners and sailing vessels, to barges, life rafts, pontoons, dump scows and even rowing skiffs.

Storage Structures include water towers and tanks, gas-meters, grain elevators, ore and coal bins, stone and sand bins, and bins, hoppers and tanks for many kinds of chemicals, solid and liquid materials.

Aqueducts include pipe-lines for water pressure or supply, sewage disposal, canals, lock gates, flumes, etc.

Subways for urban traffic are built with steel framework and steel station structures.

Tunnels have steel or cast-iron linings and shaft linings.

Contractors' plant and temporary mechanical contrivances comprise falsework, and travelers for great bridges, derricks and derrick cars, beams and girders for underpinning and temporary supports, centering for arches, forms, towers and chutes for concrete construction, coffer-dams, caissons, and framing for numerous purposes.

Miscellaneous Structures include fortifications, cars, dry-docks, piers, towers for various purposes, dams, weirs, smoke-stacks, and many unclassified structures.

Bridges.

In most cases the life of a bridge is determined by the life of its floor, and the life of the floor is dependent chiefly on design, maintenance and traffic conditions, loads often increasing far beyond the provision of the design. Accidents, overloads, corrosion and neglect are the principal factors inimical to bridge life.

Buildings.

Very few conclusive data are available of the limits of life of steel frames of skyscrapers, hotels, theaters, churches, warehouses, stations and most lighthouses, but there appears to be no reason why the life may not often be made extremely long, practically unlimited, with the best design and maintenance, if exempt from fire and serious accident.

Buildings like auditoriums, tenements, theaters, markets, etc., are so dependent on the often changeable character of the locality that they may in a few years be abandoned for the original purposes and removed or devoted to entirely

different uses, so that they are not likely to be built or maintained for extremely long life and may not justify such construction.

Buildings like mills, shops, power and industrial plants are likely to become obsolete by great changes and improvements in the mechanical operations conducted in them and require to be taken down and replaced by up-to-date structures, so that their life is not likely to need be long.

Grandstands, sheds, stations and light portable structures are either subject to very changeable requirements or are intentionally cheap and transient, and so are not likely to develop long life or perhaps to justify it. Exposed steel and iron and sheet-metal work are likely to suffer severely from corrosion, but the greatest peril to the vital members of the structure is from fire, which it is quite possible to entirely eliminate. In most classes of buildings the life, up to probably a hundred years, is potentially limited only by controllable factors, such as design, maintenance and proper use.

Boats.

Boats have very short lives on account of the usual severity of service conditions, difficulty of thorough maintenance and liability to accident.

Storage Structures.

Storage structures when not filled with corrosive materials or subject to very severe service, as in receiving hoppers for stone, ore, etc., should have long lives, if well designed and maintained and carefully operated.

Aqueducts.

Aqueducts and the like should have indefinitely long lives when thoroughly protected from corrosion.

Subway, Tunnel and Shaft Linings.

These should be designed, constructed and maintained for perpetual service, and can probably be given practically unlimited lives.

Contractors' Plant, Etc.

Derricks, falsework, centering, concrete forms, movable towers, caissons, coffer-dams and the like are often subject to great variations and abuse of service and become soon obsolete, or are designed necessarily for temporary service once only, as

in caissons, so that it is often justifiable to design them only for very short lives, or to discard them before failure occurs.

Miscellaneous Structures.

Cars are likely to be worn out or destroyed in a few years. Smoke-stacks have very severe conditions and are always short lived. Dams, weirs, drydocks, etc., should be designed for as long life as possible, but the severe service and difficulties of maintenance and liability to corrosion and accident are likely to limit the life.

DETERIORATION OF STRUCTURES.

CLASSIFICATION OF FACTORS.

The different principal factors involved in the deterioration of steel structures may be classified as natural, artificial and dynamic.

Natural Factors.

Natural factors are those dealing with the physical properties and attributes, and include initial, mechanical, chemical, stress, conditional and accidental.

Initial factors are those inherent in the structure. The principal ones are type of design, details of design, kind of design, and quality of metal.

Mechanical factors are those developed in the fabrication and erection of the structure. The principal ones are straightening, bending, punching, imperfect and inaccurate operations, attrition and distortion.

Chemical factors are those involving any chemical change, corrosion, electrolysis, acid reactions, and solvents.

Stress factors are initial rolling, mill forging, cooling, assembling, riveting, handling, transporting, erecting and static.

Conditional factors are those due to the location, surroundings, climate, atmosphere, and include many enumerated under other classifications.

Accidental factors are those of an unusual, unforseeable, improbable character, or those due to violent and unnatural conditions, physical convulsions or extraneous disasters. They include malicious injury, mishaps of fabrication, erection or operation, falling bodies, explosions, external impact, fire,

storm, flood, earthquake, collisions, derailments, and factors enumerated in other classifications.

Artificial Factors.

Artificial factors are those not inherent in the design, fabrication and erection of the structure, and are chiefly connected with service and maintenance of the structure, involving some conditional and such accidental factors as are not due to natural phenomena, and some chemical factors. They are numerous and important; the most common include locomotive blasts, acid fumes, smoke, fire, accretions, submersion, injurious contacts, vibrations, impacts, improper loading, improper adjustments and failure of prescribed functions, neglect and improper maintenance, settlements and displacements.

Dynamic Factors.

Dynamic factors are live load, wind load, temperature stresses, reactions, reversed stresses and secondary stresses, for all of which proper provision should be made in the design.

DETERIORATING EFFECTS OF NATURAL FACTORS.

An unsuitable type of design may be entirely incapable of meeting the reasonable requirements, and will be rapidly destroyed by the regular service.

Poor details of design, such as weak members and connections that do not develop the full strength of the members, may cause local failures when most other parts of the structure are efficient.

The chemical composition of the metal and its hardness, ductility, strength and other physical qualities must be suited to the service it is intended to perform.

The metal must be uniform, must conform to high-class material requirements and be free from all flaws and imperfections that would impair it for the required use.

Mechanical Factors.

Strenuous treatment in straightening and cold bending may injure the metal by stressing it beyond the elastic limit.

Punching produces local injury in hard steel, and may injure any grade of steel if the holes are too close together or too close to the edge of the piece.

Fabricated members may be damaged by unsymmetrical operations, like punching and riveting, being performed in some places and omitted in others, thus distorting the pieces and varying their required dimensions.

Injurious, imperfect and inaccurate operations are such as finishing a forging at too high or too low a temperature, using too large or too small rivets, driving rivets carelessly, drifting badly matched holes, finishing exact lengths to wrong dimensions, and neglecting clearances.

Sometimes a play in connections causes a wearing of contact surfaces that grinds them away and reduces their strength, as of pins and rivets.

A compression member may be forced out of line by improper connections so as to develop excessive stresses; a compression member may be put in tension or a tension member in compression, or torsion may be developed by deformations produced by bad workmanship or carelessness.

Chemical Factors.

Corrosion is the most universal, injurious and difficult of prevention of all causes of deterioration. Broadly considered, it is caused by moisture and may entirely destroy the metal itself, besides bursting joints by its expansive force.

Electrolysis is practically corrosion caused or accelerated by electric action and is especially likely to attack substructures buried near high-potential electric mains and bridges used by electric cars.

Acids and solvents are liquids or gases that eat into the surface of the metal and impair its surface and strength. Liquids and fumes in industrial plants, impure water in pipes and conduits, smoke in flues and chimneys, and salts in earth in contact with substructures are among the principal factors.

Stress Factors.

Rolling-mill stresses are produced by the unequal rates of cooling of different parts of the cross-sections of hot-rolled shapes, in which unequal cooling may develop such heavy permanent internal tension as to distort the pieces and necessitate severe and injurious straightening processes, or cause the pieces to twist badly when cut.

Forging stresses may be caused by the manner of heating, hammering, pressing and cooling heavy pieces in manufacture.

Cooling stresses, often large enough to break and destroy the pieces, are due to the unequal rapidity of solidification of iron and steel castings with considerable variations of shape and mass of metal in different parts of a single piece.

Assembling stresses are due to forcing one piece to conform to another with different dimensions or shapes of corresponding parts, as in bolting a flat plate to a curved angle, or in cambering a riveted girder.

Riveting stresses may be produced by the rapid driving of a long line of unbalanced hot rivets, perhaps causing distortion that may injure the member or necessitate injurious treatment for the removal.

Handling stresses are mostly dangerous flexural or torsional stresses due to improper support of very long, wide, thin or heavy pieces, usually only partly finished, and are frequently severe enough to crack the steel.

Transportation stresses are due to improper loading, allowing the pieces to become bent or defaced, projecting pieces displaced, etc.; and like the handling stresses, are especially dangerous to columns, struts and compression members in which slight eccentricity or unbalanced strength of section is very serious.

Erection stresses are of most importance in elaborate structures like bridges, buildings and towers, and are mostly caused by heavy force applied, sometimes improperly and unduly, to force members into temporary positions, as for excess cambre. in making difficult connections, overcoming insufficient clearances, or moving heavy structures. Like the handling and transportation stresses, they are often unnoticed and undetected and may dangerously impair the strength of the structure or its members.

Static stresses are developed by the weight of the unloaded structure itself and by the necessary initial stresses sometimes required in making proper adjustments. They should all be foreseen and so provided for in the design as to produce no ill effects either alone or in combination with other stresses.

Conditional Factors.

Location may be exposed to severe winds, rains, waves, floods, tides, sand storms, tropical vegetation, ice, snow, geological movements, or depredations of animals that increase corrosion or damage the structure in various ways. Location may be so remote and inaccessible that it is difficult or impossible to provide frequent inspection or suitable maintenance and repairs, as in deserts, hostile countries, or for foundations and submerged conduits.

Surroundings may subject the structure to injurious conditions not foreseeable in design. Adjacent industrial establishments may produce corrosive fumes or cover it with dust or filth or moisture, or it may be undermined or buried or made inaccessible for repairs and maintenance by adjacent operations or new constructions.

Very severe or very changeable climate is likely to increase corrosion and produce extreme temperature distortions and stresses and may make it difficult or even impossible to maintain proper care and painting.

Atmospheric conditions are important because they generally can not be avoided and constantly affect every exterior and interior portion of the structure unless thoroughly submerged or buried. Moisture, salts and acids in the air are injurious to steel surfaces and to their protective coverings.

Accidental Factors.

Accidental factors, although sometimes destroying structures, are not strictly proper elements of their theoretical life, although most of them may be prevented to some extent, ameliorated or reduced by special features of design and extra care in maintenance.

DETERIORATING EFFECTS OF ARTIFICIAL FACTORS.

Locomotive blasts are very destructive to the exposed undersides of overhead bridges, and require the latter to be fully protected by shields which resist both attrition and chemical action.

Acid fumes are chiefly important in industrial establishments and adjacent to chemical works, and, to some extent,

where the atmosphere is unusually charged with sulphuric or carbonic acid, they accelerate corrosion and are most injurious to ceilings, roof trusses and other steelwork in buildings. Painting and covering with impervious coatings are remedial.

Smoke, especially from coal containing sulphur, is very destructive to unlined steel flues and stacks. It causes and accelerates corrosion and often eats entirely through the metal.

Fire is a great menace to buildings, storage structures and sometimes to bridges. The direct action from flames, even of a small and short fire, may soften a structural steel member so much as to cripple it and wreck the entire structure. Compression members softened by heat will completely fail, tension members will stretch so as to discharge their loads, and the bending distortion and displacements of other parts of the structure may develop fatally destructive and unbalanced forces.

Accretions of filth, rubbish, vegetable, mineral and even animal nature may be so great as to destroy the functions of a structure like a pipe or boiler tube, may set up injurious galvanic action, or may induce dangerous corrosion.

Submersion in fresh or salt water, sewage or other liquid, especially when intermittent, greatly accelerates corrosion, and frequently entirely destroys thin steel.

Contact of superimposed surfaces of steel or iron, as in riveted joints or connections between members, or contact of steel surfaces with other substances, like timber, earth, rubbish, decaying matter or miscellaneous materials, is very injurious because of the opportunity for the collection and retention of moisture and consequent increased corrosion. Contact of different kinds of steel and of steel with other metals is likely to produce galvanic action with corrosion or electrolysis.

Vibrations may be caused by revolving or reciprocating machinery connected with the structure, like printing presses or engines in buildings, or locomotives and trains on bridges; or by any regular rhythmic action, like soldiers marching; or even by natural forces like wind and water; or they may be of a secondary nature, inducted from an adjacent structure in contact. Severe cases are likely to be cumulative, sometimes

attaining greatly magnified amplitude and causing sudden, violent wrecks, and oftener causing gradual attrition and distortion of wearing surfaces, destruction of rivets and failure of connections.

Impact is often unavoidable and provided for in the design, especially in bridges, which are always subject to irregularities of wheels and track, in hydraulic and hydrostatic conduits in which the movement of incompressible fluids is liable to sudden variations, and in cars, hoisting apparatus and other moving structures. Impact stresses are much greater than those from the same fixed or gently moving loads, and are likely to be very dangerous to connections and to members directly exposed to such stresses. Their effect is inverse to the mass of the structure affected. Opinions differ as to the effect of multiplied impacts or continued vibrations producing structural change and crystallization in steel.

Improper loads are deliberately applied loads for which the structure is not designed, like unusually heavy trucks or road rollers on light country bridges, high-speed or extremely heavy locomotives on railroad bridges, dense crowds of people on ordinary floors, heavy machines or safes between the beams of floors, concentrated loads on tunnel linings or steel conduits, bending loads on columns, eccentric loads on foundations, etc.—all of which may injure, cripple or destroy the structure.

Improper adjustments by means of wedging, screwing or blocking certain portions of a structure may cause entirely new and dangerous distribution and development of stresses, distort the structure or radically change its functions, as in releasing or overloading truss members, jacking up supports, increasing camber, altering continuous-girder supports and thus changing points of contra-flexure, making continuous floor stringers develop arch action, or putting flexure into heavily stressed compression members.

Failure of prescribed functions is due to improper design, carelessness, neglect or poor maintenance. Rusting, wedging, cutting, friction, obstructions and other reasons may prevent the proper movement in expansion joints and develop serious, injurious or destructive forces in the structure itself or in its

supports, sometimes wrecking the former by the destruction of the latter. Hinge bearings in arches may become fixed and greatly modify prime stresses. Especial care should therefore be taken in both construction and maintenance of all details and parts for which it is essential to provide either free movement or absolutely fixed position.

Neglect of structures inevitably causes their deterioration and ultimate destruction, unless they are of such a nature that they have great excess strength, are perfectly protected and are beyond possibility of disturbance or changed conditions, as may sometimes be the case in buried foundations, tunnel linings, conduits or low-stressed heavy structures solidly embedded in concrete or its equivalent. Important results of neglect are corrosion, worn joints, leaks, rivet failures, deflections, distortions, displacements, successive and progressive failures, cracks and fractures, all of which throw constantly increasing duties on other parts of the structure. Thorough, systematic, periodical inspection and records, both of the structure and of the operations and results of maintenance and repairs, are necessary to safeguard against neglect and improper maintenance.

Improper maintenance consists in doing the wrong thing, in omitting the right thing and in doing the right thing in the wrong way or at the wrong time. The maintenance of a structure should be under as careful and continuous expert engineering direction as its original design, construction and erection. It should comprise skilled, periodical inspection and supervision of cleaning, protection, repairs and minor renewals, and the tabulation and analysis of records of the same.

The items of improper maintenance are too numerous and detailed for enumeration here. Important ones are painting over rust, loose paint, grease, dirt, scale and on moist surface; use of poor paint; failure to clean and paint in pockets; neglect to repair defective rivets and tighten and lock nuts; permitting members to be over-stressed or under-stressed; disregarding buckled members; permitting accumulations of dirt and rubbish; permitting dangerous corrosion, distortion, displacement and vibration and impact from imperfect conditions or undue service; unsafe methods of important repairs and renewals;

reinforcing weak members with new materials of different quality so as to derange the distribution of stress, and permitting injured or defective members or material to remain in service.

Settlements are likely to cause severe and dangerous stresses, and if large enough may threaten the stability of the structure and impair or even destroy its efficiency. They are likely to be continuous, and sometimes increasing, and should receive immediate attention.

Displacements may be sudden, through accident, or gradual from slow movements or continued external forces. They are liable to injure the structure by distortion, to develop injurious stresses, and may interfere with the functions of the structure. They should be promptly investigated and accurately measured, recorded, their causes removed, and repairs and adjustments made if necessary.

DYNAMIC FACTORS.

Dynamic factors are obvious and impressive for most structures, especially for bridges, buildings, ships, towers, aqueducts, drydocks, derricks and trussed structures of varied kinds. They are necessarily involved in any pretense of mathematical computation or technical design, and thus generally receive more or less rational attention, which is indispensable, since they determine the essence of the structure; and it is often designed with sole reference to them.

Live load stresses are those due to either static or moving vertical loads imposed on the structure, and include snow loads.

Wind stresses are those produced by both static and moving wind pressures in any direction, even including vertical in some cases.

Temperature stresses are those produced by the changes of length caused by expansion and contraction due to heat and cold.

Reaction stresses are those directly produced at the point of application of loads and supports.

Reversed stresses occur where the same member is successively subjected to tension and compression.

Secondary stresses are those independent of the main function of the member, as the flexure stress developed in a long heavy horizontal compression or tension member.

Bridges and viaducts are also subject to live-load centrifugal stresses due to the movements of trains on horizontal curves, and to braking stresses due to the inertia of a train stopped or retarded on the structure.

All live-load stresses are increased by very rapid application of their loads and by application with impact.

Deliberate live-load stresses and temperature stresses may be accurately calculated, but assumptions must be made for unusual live loads and wind loads, for increasing traffic, deterioration of structure, imperfect application of loads and for reasonable and probable contingencies and variations. If these are all properly applied and analyzed and computations and calculations properly and accurately made, most structures can be so proportioned and detailed that if a proper type of structure is selected and the different kinds of stresses kept within proper intensities for the kind of work performed, the character of the member and its sectional area and connections, the indefinite application of these stresses should never deteriorate the structure.

PRESERVATION OF STRUCTURES.

A consideration of the factors which cause deterioration of steel structures will show which are the dangerous and injurious elements; if these are eliminated, counteracted or controlled, the longest possible life will be attained.

This is largely attained by excellence in design, material, fabrication, erection and maintenance and accuracy of assumption and computation—all of which have been so far perfected that, with the best existing skill of specialists and first-class construction resources, unhampered by cost restrictions, it is possible to attain for many classes of structures long, and even indefinite life, although such may not be commercially justified. These results are, however, subject to modification by climate, surroundings, atmospheric conditions, natural phenomena, accidents and to varying degrees by unavoidable deterioration of materials.

Bad design or material, injury of metal by overstress of any kind, and neglect of the finished structure, are the most serious faults. For existing structures the most important requirements are prevention of overstressing and corrosion.

Overstressing, except from dead and wind loads and temperature—which, taken alone, are usually within safe limits—generally can be controlled by limiting the service.

Corrosion can be prevented where it is possible to maintain a coating of good paint or other protection against moisture, such as a cement wash or dense concrete casing. The latter, or its equivalent, or a non-combustible shell enclosing an insulating air space is an efficient protection against fire and is indispensable for buildings and many other steel frameworks.

Paint must be well applied, elastic, waterproof and durable, and must be applied to the clean dry surface of the metal, from which rust, dirt, grease, scale, etc., should be removed, if necessary, by wire brushing, acid bath, scraping or sand blast according to the character and condition of the surface. Galvanizing, dipping, baking, hardening, and other special processes are also useful, under various conditions, to protect steel from corrosion. In some cases, as at the bases of the columns of steel frame buildings, insulation has been provided to prevent the transmission of electric currents that it was feared would cause electrolysis.

EXPERIENCE RECORDS AND PRACTICE AND OPINIONS OF EXPERTS.

The writer requested data on most of the factors already enumerated, and expressions of opinion and results of experience from about a hundred eminent engineers, most of them structural, bridge and railroad or construction specialists; and although the time was so short and the labor involved considerable, replies were received from which the following pages are either quoted or condensed. They should, even when in a somewhat informal or casual shape, be very highly valued as expressing the convictions formed from long experience, study and observation, and the crystallization of many perplexingly diverse, incomplete, and even contradictory facts to harmonious

conclusions and practical, workable results, that could never be attained by mathematical analyses and equations. More of such invaluable data would doubtless have been secured if the time had been longer and if it had not been considered too onerous to urge such laborious work and demand so much valuable time as they necessitated from less intimate friends and from mere acquaintances. Those who have so ably and generously contributed from their store of hard-gained information will doubtless feel pleased if their action inspires and facilitates a sustained investigation of the subject commensurate with its importance and the scope indicated by the outline here suggested.

CORROSION.

Mr. Charles Evan Fowler.—"The writer has never known of any cases, in his thirty years' practice, where corrosion has gone on to such an extent as to render a structure absolutely unsafe—unless it has been due to the corrosive action of gases from locomotives or otherwise, or from gross carelessness in allowing dirt to collect in pockets, or in allowing rusting to take place where wooden fillers were used, or where the bottoms of columns were buried in dirt.

"The rusting of steel is really a very valuable adjunct in the preservation of steel structures, if stopped at the proper time when all the mill scale has been loosened. The structure then should be thoroughly cleaned with sand blast and painted immediately with some paint of a red lead, litharge, or Portland cement base; this method could easily be observed around any yard where a stock of structural steel is carried, and where the scale rusts loose in the piles.

"This method has been used to some extent in English railways, where the method of the Manchester, Sheffield and Lincolnshire Railway is used, that is, no part of the structure is to be painted before erection, except such portions as are inaccessible for painting afterwards. After the structure has been erected and the scale has disappeared—owing to the rusting process—then the metal is thoroughly cleaned and the paint applied.

"The most common cases of corrosion or rusting, to in any wise near a dangerous extent, in bridges occur where there are pockets or places where water can stand, or which are inaccessible after the structure is erected. All such places should be, as far as possible, filled with concrete, or better still, be avoided in the design. In city bridges, where the floor comes up around the web members tightly, there is nearly always a badly rusted ring formed; such places should be avoided by leaving an air space and making a curb around the opening."

In the Proceedings of the Pacific Northwest Society of Engineers for Dec. 1902 there is published a paper, "Corrosion of Iron and Steel", by Mr. Fowler, then president and chief engineer of the Puget Sound Bridge and Dredging Co. It states that iron usually rusts in a uniform manner, the rust being directly a part of the metal, while steel rusts chiefly between the scale and the body of the metal proper, so that when a piece is picked up and dropped again great flakes, seemingly all rust, fall off, which in reality are the scale rusted loose. It is the writer's opinion that iron that has become badly rusted in storage is seldom, if ever, entirely cured by any protective coating that may be added. On the other hand, it is seemingly a good thing that steel should rust, and in rusting thoroughly remove the scale, although the rust directly upon the surface of the metal may be hard to remove or kill. It is not, however, necessary or desirable to store steel for the purpose of removing the scale in this way, as most of it flakes off during the process of manufacture—that is, while straightening, punching, reaming, and riveting—to such an extent, at least, that it is doubtful whether it is desirable to paint or oil raw material before fabrication, as some specifications require.

It is usual in designing structures which are subject to excessive corrosion to add a certain percentage in the thickness of the metal to allow for deterioration, this thickness being in excess of that necessary to carry the load; and this certainly should not be overlooked by engineers using metal where excessive deterioration is probable.

Rusting, corrosion and pitting are the great causes of the deterioration of steel ships. Rusting occurs most rapidly near

the water line, where the metal is subjected to the combined action of salt water and the air. The pitting is usually the result of galvanic action, sometimes extending over areas as large as 1½ square feet. To properly care for a ship's bottom, the ship should be docked every six months or so and should never be allowed to go longer than a year. It has been stated to the writer by Mr. Frank W. Hibbs, Naval Constructor, U. S. Navy, that steel ships corrode faster than iron ships in the ratio of three to two, and I presume this is the result of records of the U. S. Navy, and of Mr. Hibbs' own observation. The Chief Engineer-Surveyor for Lloyds, who has had the best of chances for observations, states that steel does not corrode any faster than wrought iron.

A great quantity of metal has been dredged up from the harbor of Brest, France. The oldest cast-iron dredged up was in the form of cannon balls and shells, the latter once filled with powder. They dated from 1652 to 1791, and thus had been immersed from 100 to 240 years. These cast-iron objects were all encased in a hard coating several centimeters thick, made up of sulphurets, sand, rust and calcareous concretions. This coating required a hammer and chisel to break it, and came off like a mould, leaving the objects encased little altered in appearance and simply covered with a viscid, black layer, disagreeable to the smell. It was a mixture of hydrogen sulphide and silica.

This iron had, however, been subjected to very considerable alteration, as revealed by its light weight. All specimens had undergone to a greater or less extent that type of decomposition known as the softening of cast iron; and M. Lidy believes that the objects examined by him prove, beyond any doubt, that such softening has here taken place. Specimens could be cut with a knife as easily as the graphite in a pencil and showed a similar brilliant black section, which tarnished quickly upon exposure to the air. It broke easily and was quickly reduced to powder in a mortar; it could be readily sawed, and when struck with a hammer a dull surface was obtained covered with a multitude of small brilliant points.

Exact analysis is impossible, owing to the lack of homogeneity in the material and the fact that it oxidizes very rapidly on exposure to the air. As a result of analysis, M. Lidy con-

cludes that this iron originally contained about 0.933 iron and 0.067 foreign matter; of which latter, 0.054 was graphite and 0.010 silica. Comparison, on this basis, shows that the object had lost about 69 percent of its original total of iron.

As a general result of his investigations, M. Lidy concludes that, exposed to the action of sea water, cast and wrought iron are rapidly decomposed, not only upon the surface, but in the interior of their mass. In wrought iron the superficial action is much the stronger; while in cast iron it is the internal action which predominates; and this is peculiarly dangerous because the form of the object is not affected, while the resistance is considerably diminished. M. Lidy believes that the necessity is shown for testing, at intervals, all cast iron exposed to the sea, so as to be certain of its preservation and strength to sustain the loads placed upon it.

Mr. F. R. Harris.—"On steel fastenings for wharf work, I have found that galvanizing for under-water work does not greatly retard corrosion. Corrosion seems to be more emphasized near the surface of the water. The entire destruction of fastenings of average sizes, however, consumes considerable time. It seems to be well worth while, where possible, to use composition metal, such as extruded brass, or to as much as possible depend on tree-nails and fitting of timber. Where the fastenings are covered or well surrounded, even though below tide level, incipient corrosion seems to be rapid, but after this has taken place, very little or no progress of corrosion appears to occur. I have found wall anchor rods thirty years old only superficially affected."

Mr. F. W. Hibbs.—"Iron ships seldom give much trouble or concern, and if it were not for the extra thickness required for equal strength and the difficulty of obtaining large iron plates, with the consequent extra cost of building and loss of carrying capacity, there would be no question of the advisability of plating a steel ship's bottom with iron. I would have no fear for the bottom of an iron ship if it were to remain in the water for a year or more; in fact, I have seen cases where iron vessels were not docked for nearly two years without any serious re-

sults; but a steel ship will invariably show serious corrosion at the end of eight months, and should never be allowed to go more than six months.

"As regards corrosion, the application of steel to ship construction is probably more severe than any other structural use to which it is put; not only because it is of necessity more exposed to the action of the elements, and particularly to the very active agent, salt water, but also because from the very nature of the case many portions of the hull are either so constructed as to be inaccessible, or are rendered so by the service of the vessel.

"The most constant care is necessary for the preservation of a steel ship; and this is not merely a theoretical idea, but a practical fact; for it may easily depend upon the care which it receives, in this regard, whether the vessel lasts for active service fifteen years or forty.

"The parts of a steel hull which are most liable to deterioration are the bilges, the fore and after peaks, the waterways, ballast tanks, shaft tunnels, rudder port, rudder, and around the boiler fidleys, hatch coamings, scuppers, and hawse pipes; and on the bottom, along the water-line, in the entrance and run, and in the neighborhood of sea connections and ash chutes.

"The bilges of the boiler and engine rooms, particularly the former, are probably the most troublesome. Here the bilge-water, from which these parts of the ship are never free, composed of salt water to begin with and containing ashes, coal, and acids from decomposing lubricating oils—all washing about with the motion of the ship, and in the presence of heat and moist air—soon destroys the protecting paint and attacks the steel work. Moreover, corrosion goes on the more rapidly as it is impossible to maintain these bilges clean or even dry while the ship is under way.

"It is customary in these spaces to protect the steel, as much as possible, with Portland cement, worked from two to four inches thick; and this is perfectly effective so long as the cement remains intact; but the lower portions of the bulkheads, the boiler saddles, and ships without a double bottom, which

cannot be thus protected, usually become very much corroded. In ships with wing bilges (ballast-tank construction), this condition is very much improved, because more of the steel is protected with cement, and there is no opportunity for the bilge-water to lodge amidships.

"The tops of the ballast tanks frequently become much corroded in the hold spaces, on account of being entirely covered with tight wooden ceiling. From the nature of things, it is not possible to raise this ceiling very often, and any water in this confined space . . . soon corrodes the steel. The peaks are usually filled with concrete to such a height that these spaces can be cleaned and painted, and, in general, it is a good rule for the preservation of the ship to fill up all pockets or spaces that can not readily be approached for cleansing and painting.

"As an example of the extent to which corrosion may go, I will mention the case of a coasting steamer, built without a double bottom, which had been in service about twelve years, in which, solely from lack of attention, the boiler saddles, floor-plates of the frames, keelson plates, and even the keel plates in the boiler room, were eaten away by the action of bilge-water to such an extent as to leave nothing but the angle-bars in some places. An average of twenty percent of their area was entirely gone through, and the steel remaining was so thin that it could be bent with a hammer. The action was hastened by the heat from the boilers, and the fact that the bilge spaces were closed in tight, with little opportunity for the circulation of air."

Mr. A. F. Robinson.—"Regarding corrosion (of bridges), I may note that I have found it possible by the use of the high-grade paints to obtain a life of six to ten years for the second coat before it needs repainting, while the body, or first coat, is always left covered and is never influenced by the atmospheric condition. When metal bridges are handled in this manner there is no reason why the metal work cannot be protected permanently from corrosion."

Mr. J. P. Snow.—"Corrosion is an active agent, of course, in putting steel bridges out of use. The most active agent in

producing corrosion that I have been up against is locomotive gas. The character of coal used has a great influence on the energy of the resulting corrosion. Coal or coke containing sulphur is much more destructive than that which is free from this element. The effect of gases is much greater on bridge steel than upon wrought iron.''

Mr. J. A. L. Waddell.—''I believe that it is perfectly feasible to take care of metal from the moment it is rolled until it is erected and painted, in such a manner as to prevent any corrosion worth mentioning setting in. Again, I believe it is practicable, by proper care, to prevent a steel bridge from ever rusting. Again, I am confident that a bridge properly protected will last for an indefinite length of time.''

Mr. Aubrey Weymouth.—''So far as my personal experience is concerned, corrosion is the most important thing to be guarded against. I recall one very bad case of corrosion in grillage girders erected in a high building down-town of twenty years ago. In those days I think the builders were not so careful to protect grillage girders with concrete, and the result was that these girders, when exposed during the construction of an adjoining building, were found to be badly corroded.

''The second case of corrosion that I recall is in a steel stack on a building the height of which we were increasing several stories. It seems, however, that this corrosion from the gases is limited to the upper three or four feet of the stack, and when this portion was cut away the remainder was found to be in a very good condition. It would seem to be good practice, therefore, to make the top section of the stack somewhat thicker to allow for deterioration, although this is not the general practice.''

Mr. O. H. Ammann.—''There is at present no commercial means of preventing the rusting of steel structures and thus prolonging their lives, except by keeping the steel constantly well painted, and if this is done properly there is no indication why the steel should not last indefinitely. It depends largely upon local conditions, particularly on the presence of acid gases or sea water, as to how frequently the paint has to be renewed.''

Mr. John C. Naegeley.—"If the steel is of proper quality, well finished at the rolling mills and carefully treated at the structural shops, particularly in the matter of oiling and painting in shop and yard, which is seriously neglected in some shops, the conditions which cause corrosion will be to a great extent prevented."

In a paper on Protection of Metal Structures, published in the Proceedings of the Engineers' Society of Western Pennsylvania for March, 1915, Mr. F. H. Fay, consulting engineer, Boston, and formerly engineer of bridges for the city of Boston, says that the corrosion of Boston bridges has been principally due to (1) exposure to locomotive gases; (2) exposure to sea water; (3) exposure to surface water leaking through bridge floors; (4) over-stress of the metal, by which corrosion has been hastened.

Concrete has proved to be by far the most satisfactory protection for metal structures over railroads.

In the badly corroded and rebuilt Boylston Street Bridge over railroad tracks, a cast-iron fence-post base was found almost intact while the wrought-iron bracket supporting it was almost destroyed by corrosion.

It is probably possible to build concrete that will be durable under alternate exposures to air and salt water.

Cushman and Gardner in "Corrosion and Preservation of Iron and Steel" state that mill scale produced electrical action causing rust.

Chilled or hardened steel and steel stressed above the elastic limit are said to have increased resistance to rust.

The ten-story Tower Building, on Broadway, New York, although only 119 feet high, was very narrow and slender in appearance and was one of the first of its class to be erected, and being conspicuous was commonly called the first skyscraper. All columns were of cast iron, 20 inches square and about 1½ inches thick, and were fireproofed by an outer cast-iron shell ⅜ inch thick. The wrought-iron floor beams carried flat terra-cotta arches, which did not protect the tops and bottoms of the beam flanges. The beams were, in the upper five stories, supported on the exterior walls; below that level

they were carried by wall girders, and other girders supported the brick walls. All field connections were made with wrought-iron bolts and all shop connections were riveted. The steel was apparently painted with a metallic and linseed-oil paint.

Bureau of Buildings, Borough of Manhattan, New York, publishes in its 1914 report several photographs, showing the condition of the structure when demolished.

"One photograph shows a double exterior wall column taken at the level of the fourth story. The outer shell, especially near the foot, was slightly covered with rust. Rust spots were also beginning to form on the surface of the inner column, probably due to moisture percolating through some loose joint. A number of these columns were examined and found to be almost devoid of rust.

"Another photograph shows the lower portion of a cast-iron column in the basement, and to which had been connected the wind bracing. The formation of rust at and around the bolts can be readily distinguished. This may be cited as the worst case of rusting in the Tower Building, and exemplifies the fact that corrosion takes place readily at connections where two surfaces come in contact.

"Another photograph shows the connecting angles, the surface of which was entirely covered with rust, but not in quantity sufficient to materially reduce the strength of the connection.

"A steel column and girder in the exterior wall of the adjoining building (Exchange Court), which was uncovered during the demolition, showed a large rust scale about 1-16 inch thick which had formed on the underside of the column plate. The surfaces of the steel girder are also seen to be badly pitted in places. The severe corrosion at this point is probably due to defective flashing on the roof of the Tower Building, permitting water to percolate through and enter the vacant space between the independent walls.

"In a riveted column in the wall of Exchange Court Building, almost the entire surface, rivet heads and all, was covered with rust.

"After a most careful and thorough examination of the iron work in this building, it is safe to say that there was

practically no case of corrosion sufficiently advanced to impair the strength of the building. The following conclusions seem warranted:

"Steel and wrought iron are more susceptible to corrosion than cast iron.

"Exterior columns and girders, because of their location, are more apt to become affected with rust than interior ones.

"Rust forms readily at connections where two surfaces come in contact.

"Beams resting on brick or stone work usually corrode near the ends.

"If not properly cleaned of rust and scales, iron will corrode underneath the paint. The nature of paint used is of no greater importance than the manner in which it is applied.

"Cement mortar appears to neutralize the action of rust. When in close contact with the metal, cement mortar forms the best protection.

"Paint with linseed oil as a vehicle saponifies when in contact with cement, leaving the pigment dry and incohesive. It appears therefore to be of little importance whether the steelwork is painted or not, provided the metal is thoroughly encased in mortar or concrete".

The 16-story steel-cage Gillender Building, New York, was built in 1896 and demolished in 1910. The structural steel was painted with two or three coats of metallic linseed oil paint, and the columns were encased in brickwork with rich cement mortar fill that became so hard it was removed by pneumatic tools.

The paint had all disappeared where in contact with the mortar, and wherever there was poor contact between mortar and steel, rust had formed in small quantities. No rust had formed where there was mortar protection. The under sides of the beam flanges had commenced to rust. The worst corrosion was on columns at the southeast corner, where moisture penetrated the brickwork.

Mr. A. W. Carpenter reported that "While some of the steelwork showed slight corrosion and pitting, there was noth-

ing to cause any special apprehension regarding the ability of such buildings to endure for an indefinite term of years.

"A covering of cement mortar protects steel from corrosive influences better than any form of paint at present in use.

"It is important to paint the steel, both at the mill and after being erected at the building, before the cement covering is applied."

In a discussion published in Vol. L of the Proceedings of the American Society of Civil Engineers, Mr. Rudolph Miller described the condition of the Pabst Building, New York. It was a steel-frame building completed in 1899 and demolished in 1903. It had received one shop coat of carbon paint and some parts had not been repainted in the field. The interior columns were fireproofed with terra-cotta enclosing an air space, and were very little rusted. The heads of rivets and connections were the most rusted, but the rust was hardly measurable. The wall columns were encased in brick and were only slightly rusted. The floor beams enclosed in 1:2:5 cinder concrete were remarkably free from rust. The building was erected in very wet weather, and the bricked-in wall girders were rusted on the outer flanges. The smoke stack was covered with rust, although the wire reinforcement in its asbestos jacket was bright.

The records of the Bureau of Buildings, New York, state that the steel-frame Hoffman House and Albemarle Hotel were completed in December, 1907, and demolished in July, 1915. They had brick walls, terra-cotta arch floors and cinder-concrete fill, terra-cotta jackets for interior columns and 4- and 8-inch brickwork around exterior columns. The structural steelwork was painted two coats and "was in most places intact without the slightest sign of corrosion". The worst case of rust was of columns in exposed outer wall where moisture collected in open spaces between face bricks and brick backing. The worst rusting was at and near connections, but was not bad enough to impair the strength.

Sheet piles driven in very wet ground around the foundations of the Hoffman House were found unrusted when removed eight years later.

The records of the Bureau of Buildings, New York, describe cast-iron columns enclosed in brickwork and supporting coal bunkers above the boilers in a boiler house on 55th Street, which after 25 years' service were found to be badly rusted to a maximum depth of ½ inch for a distance of 4 feet above the bases. This was considered due to the penetration of steam through the porous brickwork, as the exposed upper portions of the columns which were painted were not rusted.

A paper presented by Mr. Clifford Older to the Illinois Society of Engineers and Surveyors in 1914 states that in Illinois serious damage from corrosion results from neglect to paint highway steel bridges after their acceptance. This is aggravated by the use of salt to thaw the snow and ice and by the appreciable amount of acid contained in the soil in many places, which accelerates the corrosion wherever dirt lodges in contact with the steel and causes the complete destruction of ¼-inch plates in ten or twelve years. The initial coat of paint affords protection for only two or three years. In five years of service the horizontal flanges of 4 x 6 x ½-inch girder lower-chord angles which support wooden floor beams were entirely destroyed by corrosion.

Frequently the formation of rust between the cover plates and flanges of compression members, swelling to about double the original volume of the metal, develops sufficient force to open cracks ¼ inch thick and buckle the cover plates between rivets.

A large proportion of the rivets in the ribs of a 234-ft. wrought-iron arch span, built in 1869 across the Fox River at Ottawa, Illinois, were destroyed by corrosion, which expanded the riveted joints so as to break the rivet shanks by tension or pull off their heads, although the bridge had been kept reasonably well painted.

A 120-ft. plate girder span in Boston was kept in service until the web plates were full of large and small holes and the remaining metal was so much softened that much of it could be knocked out with a small hammer.

The U. S. Lightship 71, which was built in 1897, was anchored off Cape Hatteras, and after 11 years was found to have had 8400 4-inch bolts destroyed by electrolysis.

ELECTROLYSIS.

Mr. G. A. Orrok.—"I have only a few examples of the effect of electrolysis in mind, as most of the electrolytical actions which have been reported are in my opinion not electrolytical, but simply chemical. As to the chemical action of water, moisture and other things on steel, of course, we have many examples. To put it into few words, I have never had any cases of serious steel corrosion which could not be traced to ordinary simple chemical reasons and prevented by the proper precautions against moisture".

Mr. O. H. Ammann.—"Electrolytic corrosion of steel is an established fact; it is particularly active where steel is in contact with water or humid air. The corrosion always takes place at the anode. This action can however be prevented by keeping the steel well painted. Certain paints are claimed to be especially adapted for protection against electrolytic corrosion, but there is as yet not sufficient proof available".

Mr. C. E. Fowler.—"Electrolysis, while not an ordinary form of corrosion, should be mentioned briefly, as it is of great interest to city engineers in particular. Where the return circuit of an electric railway is made through the rails, considerable current leaks out where the bonding is poor".

Mr. J. A. L. Waddell.—"I am not sufficiently conversant with this matter to give information of much value. Certainly every precaution should be taken to protect metal-work on bridges from electrolysis".

Mr. A. W. Buel.—"As confirming the theory of electrolysis as caused by the magnesia content of steel, I might refer to the fact that old wrought-iron bridges built thirty or forty years ago in Ocean County, New Jersey, were still in fairly good condition after steel bridges of much more recent date had nearly rusted away. You may also look up the remarkable preservation of the wrought-iron links of the Newburyport, Mass., suspension bridge, built in 1810".

Mr. F. R. Harris.—"There have been some discussions on the theoretical phase of corrosion in the Naval Institute, the latest and most accepted being that it is due to electrolytic

action, depending upon the difference of potentials in various parts of the plate or structural work, perhaps traceable to inherent differential stressed condition of the metal, brought about in process of rolling or fabrication, electrical interchange occurring through the salt water as a medium, the impurities in the iron or steel figuring largely in the corrosive tendency of the material''.

An article published in the Engineering Record of January 8, 1910, says that the track spikes in a railroad bridge penetrated through the ties about to the surfaces of the stringer flanges, eventually making contact in many places and causing a heavy leakage of electricity, sufficient to char the wood and make holes, some of them extending entirely through the flanges. One of them had a diameter of 2 inches and a long steel stalactite depending from it. There were many shorter stalactites and many of the spikes had their lower parts eaten off. Computations showed that the injury to the stringers reduced the safe loads from 50 tons to 43 tons. A careful examination of the bridge two years before this trouble was discovered disclosed no indications of deterioration.

ALTERNATE WETTING AND DRYING.

Mr. J. P. Snow.—''I have had considerable experience with steel and iron-work submerged in brackish sea water. This went under water twice a day with each tide. It was exposed to the air somewhat longer than it was submerged. In this situation there was but little difference in durability between iron and steel. Neither corroded so rapidly as steel exposed to engine gases''.

Mr. Jonathan Jones.—''I have seen galvanized steel sheeting rot out in a very few years where a bad detail around a window kept water lying on it after a rain, whereas those portions which were quickly drained showed little deterioration in the same time''.

Mr. J. A. L. Waddell.—''Alternate wetting and drying deteriorates metal most rapidly. From what I can learn, steel constantly submerged in salt water may deteriorate more or less rapidly, depending upon the temperature of the water; but

that constantly submerged in fresh water is attacked very slowly, if at all."

PREPARING STEEL SURFACE FOR PROTECTION.

In order to insure proper adhesion and efficiency of the protecting film, it is necessary to have the surface of iron or steel perfectly dry and clean and free from rust or scale. Painting, galvanizing, dipping, etc., therefore, should be, and usually are, preceded by washing, brushing, scraping, pickling or sand blasting the metal and sometimes by heating it.

Professor C. D. Marx.—"Between 1905 and 1907 the Pacific Improvement Co. laid a new 22-inch steel main at Monterey, California. Mr. Thomas B. Hunter was engineer in charge and I acted as consulting engineer. The specifications as drawn up contained the following clause:

> All plates and rivets must be free from rust, and be kept under cover from the time of manufacture of the plates until the completed pipe is dipped. In case of accidental rusting during transportation, or manufacture of the plates, or manufacture of the pipe, the rust must be at once removed by brushing with stiff brushes and scrubbing with dilute acid, following by mopping or brushing with saturated solution of soda, or suitable alkali to remove the acid. The alkali must then be thoroughly washed off and the plate or pipe thoroughly dried.

"The method prescribed for cleaning off rust was tried. The cleaning with wire brushes is not satisfactory, as it will not take out all the rust from "pitted" spots, and in the main, takes off only the surface of the rust. In other cases, the rusty steel was swabbed with dilute muriatic acid, and afterwards neutralized with slaked lime solution. In spite of all precautions of washing off the lime and then drying, the rusting took place faster after the treatment than before, and even some of the original rust remained in places when exceptionally thick. The method of dipping in a pickling vat was not tried because of lack of facilities.

"It was afterwards found that dilute sulphuric acid would dissolve the rust very satisfactorily.

"At the time of the great fire of April 18, 1906, all the steel necessary to finish the contract was on hand. The piles of steel, a carload or more in a pile, evidently did not heat

through, though the top sheets and the edges of the projecting sheets were burnt and warped. No water whatever was thrown on the shop and contents at any stage of the fire.

"However, the piles remained without protection from the rains for several months, pending reconstruction of the works, and were badly damaged by rust. The firm purchased a sand-blast outfit at the request of the inspector, and about 3000 sheets were cleaned perfectly by this means. A few notes on this method may not be amiss.

"When kept indoors after cleaning, provided an efficient water trap is used with the machine, steel cleaned by sand-blast method will not become seriously rusted for from one to four weeks or more, depending upon the humidity.

"Air pressure of 40 lbs. per sq. in. was found by experience to be the most efficient for steel, all things considered. The machine was a 16-inch Buckeye. It was originally a single-nozzle machine, using a large sand hose, $1\frac{1}{4}$ inch. By having a Y pipe made, it was arranged for a double nozzle, and ordinary $\frac{3}{4}$-inch garden hose was used, with home-made nozzle tips and holders.

"The light hose is more easily manipulated than the large stiff hose. The nozzle tips gave much trouble. Case-hardened steel, cast iron, semi-steel, rubber, white iron and hardened tool steel were used, the latter giving the best satisfaction. The opening through these tips was $\frac{3}{16}$ inch.

"The machine used about 100 cu. ft. of free air per minute at 40 lbs. pressure. The capacity of the sand tank was 3 cu. ft. but should be greater, as too many stops are necessary for filling. The sand should be dry, free from dirt and slivers, and should be sized through a 10-mesh screen. It should be hard quartz if possible.

"Some means of catching the sand should be provided, as it can be used several times over before it becomes too finely pulverized. Beach sand is hard enough, but the grains are too light to strike with force enough to be efficient. Sand from the Presidio Beach, San Francisco, proved to be the best on the city market, as it is coarse and sufficiently hard to produce good results.

"Two men, each using a sand hose, averaged 31 sheets cleaned on both sides per day, at a cost of labor of 25 cts. per sheet, working by piece work.

"It is of great importance that an efficient water trap be placed in the air pipe supplying the machine. If a delay is expected before the steel is dipped in the protecting coating, it is well to oil or grease the bright surfaces left by the sand-blast. This is easily done with an oily rag; but if possible, however, it should be avoided, as it makes subsequent handling in the shop very unpleasant and also often causes trouble by slipping in the curving rolls".

Mr. A. W. Buel.—"I believe in the sand blast under certain circumstances, but it is absolutely essential that the first coat of paint be applied immediately before the clean surface has lost its brightness. My experience with wire brushing is that it is almost impossible to get it done thoroughly, and if it were done thoroughly, it would be very expensive".

Mr. J. A. L. Waddell.—"If not applied too vigorously, sand blast is all right for cleaning metal work. Acid bath also may be useful in cleaning, but all acid should be removed by washing with water. Wire brushing, if applied thoroughly, is beneficial in cleaning metal, and I do not believe that the brushing has any material tendency to wear off any of the metallic surface".

Mr. C. E. Fowler.—"The lack of thorough cleaning of steel in the shop before painting is a very fruitful cause for the beginning of corrosion, and the lack of thorough cleaning of structures when they are being repainted is also another very fruitful cause of trouble. The writer has in mind one bridge, where, after only four years of service, it has rusted so as to necessitate very drastic treatment in repainting—due to the fact that it was repainted over the old paint without having been properly cleaned".

In 47 replies to a circular issued by the American Railway Engineering Association asking particulars about the use of the sand blast for cleaning iron and steel preparatory to painting it, one railroad had used it extensively and liked it; seventeen

railroads approved its use; three disapproved its use; ten considered it too expensive; one objected because it leaves a rough surface; and one disapproved because in unskilled hands it cuts the metal.

Mr. A. D. Flinn's notes on pickling large steel pipes at the works of the East Jersey Pipe Co. state that 48-inch pipes, 30 feet long, are dipped 15 to 20 minutes in wooden tanks containing 3 to 7% of 66 degrees Baume sulphuric acid at 180 degrees temperature, then washed two or three minutes in a water tank, heated in an oven and dipped in Sarco. These notes also describe the method of pickling steel plates in the U. S. Navy Yard, Brooklyn, where plates are pickled 24 hours in wooden tanks containing a solution of 230 gallons of muriatic acid to 2618 gallons of water, at 120 degrees Fah. They are then washed 20 minutes in lime water and stored until used, when the rust is brushed off. About one half of the rust can be removed and costs about ¾ cent per square foot. The pickled surfaces rusted uniformly, while unpickled angles riveted to them were very rough with small rust pits.

PAINTING AND PAINTS.

The United States ship canal at Panama is commanded by 46 pairs of hollow steel lock gates which are much the largest and most important ever constructed. They are 65 feet wide, up to 82 feet high, have a total weight of 58,582 tons, and cost more than $7,000,000. They are intended as much as possible for perpetual, uninterrupted service and no cost or pains were spared in the design, construction or methods of protection to secure for them the most complete preservation against corrosion and the longest possible life.

The gates were designed and constructed under the direction of Mr. Henry Goldmark, who was eminently qualified for the very responsible work by his previous experience in the design of large lock gates and by many years of practice as a bridge and structural engineer. The provisions adopted to prevent corrosion of steel under exceptionally severe conditions were intended to be the most efficient that could be devised, with the information and facilities available at this time to the unlimited resources of the government, and are clearly de-

scribed by Mr. Goldmark in the following statement written for this paper:

"The protection of metallic lockgates from corrosion is a matter of much importance, as the replacing of the gates is difficult and expensive. The problems encountered are more nearly those met with in ships than in bridges or similar structural work. Part or all of the exterior surfaces of the gates are immersed, either permanently or for a part of the time, in salt or fresh water.

"In gates with flotation chambers, the conditions in the interior are similar to those in the double bottoms and other small compartments in vessels.

"The entire outside of gates is accessible only when the locks are unwatered. There is no special difficulty in maintaining or renewing the exterior paint, but the interference with the use of the locks is of course undesirable and, hence, even for the outside surfaces it is important to secure a coat which will last as long as possible without renewal. However, as stated by Naval Constructor Williams, U. S. Navy (Transactions American Society Naval Engineers, May, 1912), "One practically never hears of a steel ship that has a hole in its bottom plating due to corrosion from the outside". Similarly, with ordinary care and inspection, the exterior surfaces of gates do last almost indefinitely, some gates over forty or fifty years old remaining today in perfect condition.

"On the other hand the interior surfaces of the gates are very difficult of access and in many locations the atmosphere, being hot and humid, is very severe on any coating that may be applied. It is difficult to get men to do careful work in these confined spaces and the work of supervision and inspection is also likely to be slighted.

"As a matter of fact, no one paint has proved entirely satisfactory on the interior of lock gates. Various carbon paints, bituminous coatings as well as ordinary red lead, graphite and other paints have been used with varying success.

"Much depends, of course, on the character of the water, the prevailing temperature, etc.; but even under the most favorable circumstances, frequent repainting has as a rule been necessary.

"In Panama the conditions were seen from the beginning to be unfavorable. The temperature of the air as well as the water is always high, the former rarely dropping below 70° and the latter below 80°. Skilled labor is expensive, so that repainting will always cost more than in the temperate zones. The locks are to be operated throughout the year, so that repainting would interfere seriously with traffic, although the twin system of locks would reduce this trouble somewhat.

"It was thought justifiable, even at greatly increased cost, to select a coating which promised to have the longest possible life.

"The writer's attention was directed to the so-called Bitumastic Enamels which have been used for many years for certain parts in the interior of ships, which are exposed to especially unfavorable conditions. These coatings are proprietary compounds, the exact composition of which has not been made public, although the chemical composition shows that they are coal-tar products, usually with some asphaltic mixture. They are applied *in situ* when in a hot molten condition by experts. It is required that the metal surfaces be thoroughly cleaned. The first coat, the so-called bitumastic solution, is applied cold. The enamel cannot be applied successfully over red lead or other paints.

"In view of the excellent results obtained with this coating in the United States' and other navies, also in merchant ships and in certain dredges at work on the Panama Canal, the entire interior of the lock gates was covered with bitumastic enamel. The contractors guarantee under bond to keep the interior in good condition for five years.

"This material was not specified for the outside surfaces, as it had never been used, previously, except in interior compartments. On the exterior surfaces it was intended to follow common practice and two heavy coats of red-lead paint, covered by a third coat, the nature of which was left open for further consideration, were specified.

"The conditions which were found to prevail after the locks were put into operation were in some respects more severe, and in others less so, than had been expected. The vege-

tation which was submerged in forming Gatun Lake produced large quantities of sulphurous tri-oxide (SO_3) and carbonic di-oxide (CO_2). The red-lead paints with a finishing coat of graphite paint which had been put on the gates first erected suffered severely after being immersed only two months or so. It was therefore decided to make an extended trial of several different types of paint. In the Gatun Locks all gates except those in salt water were given two coats of red-lead paint covered by two coats of a graphite paint, containing about 50% of red lead, while the lower guard gates and lower gates received a coat of U. S. N. anti-corrosive paint, over the red lead, covered by a final coat of anti-fouling paint. In Pedro Miguel Locks several coats of a well known brand of damp-proof paint were used; and at Miraflores, "Inertol", a bituminuous paint, applied cold was employed, except for the lower gates and lower guard gates, which were treated like those at Gatun. Experience up to the present time shows that none of these paints can be counted upon to give satisfactory protection below the low-water line in the water of Gatun Lake. It is understood that steps have already been taken for substituting bitumastic enamel at Gatun, as this compound has so far proved a perfect protection, both to the inside and outside of the gates. Its high price is, of course, a serious objection, but under the conditions prevailing, its adoption seems to be entirely justified.

"At the structural shop where the lock gates were fabricated great pains were taken to have the surface of the steel thoroughly clean and free from rust when painted. To this end special pickling and washing tanks were provided in which a large quantity of the separate plates and angles were treated as soon as convenient before fabrication, and an extensive sand-blasting plant was installed by which riveted portions of the gates were thoroughly cleaned at the latest practical moment before painting."

Mr. F. R. Harris.—"In the work that has come under my observation, either outside or in connection with my regular duties in the Corps of Civil Engineers of the Navy, I have no information or observation of value to give to you on structural

steel used in building construction, other than perhaps comment on our standard specification for painting, which until recently called for removal of mill scale by wire-brushing, a shop coat of red lead, and two protective coats in the field of white lead and pigment or graphite paint.

"In recent years, in instances, the shop coat has been changed, permitting the use of silica or cement paint of inert character. Recently, at the New York Yard, structural steel of the coaling plant, which had not been repainted in more than two years, showed rust marks. This plant was repainted with one coat of such paint, the paint used being "Toxement", a proprietary paint manufactured by Toch Brothers, of New York, the second coat being the same company's R. I. W. 49. Similar paint was used on the heavy cranes of the Yard.

"The Bureau of Yards and Docks at Washington has had some experience in the protection of structural steel wireless towers. In many instances these are located in tropical climates, although they have not been in existence long enough to give much data of value. The wireless station at San Juan, Porto Rico, came under my jurisdiction as Public Works Officer at New York. The painting was done with a shop coat of red lead and a finishing coat of standard torpedo-boat green, and within a year after completion the work showed rusting and pitting.

"My observations in Cuba and in other tropical locations indicate that the temperature and humidity serve to greatly accelerate corrosion of steel and iron, which made it extremely difficult to find a satisfactory protective coating. Light tools, such as iron shovels, were entirely destroyed within two years. Spiral-riveted dredge pipe, originally coated with one coat of asphalt varnish, pitted and corroded through within two years. Corrugated galvanized iron, in use for buildings, would last longer, but its life is considerably less than in the United States.

"In steel walls immersed in water, especially sea water, cases coming under my immediate observation and supervision are dry dock caissons or gates. The gate of Dry Dock No. 4 at the New York Yard was painted with red lead. Within one

year after its arrival and use in the dock, the exterior surfaces showed general corrosion, spotting and pitting through the protective painting.

"The caisson was docked, old paint-work chipped and wire-brushed and the caisson given one coat of "Toxement" and two coats of R. I. W. 49. After two years the surfaces were in fair condition, showing some corrosion. There is some acid present in the East River water. Caissons of the other three docks show slight corrosion and require repainting about every two years. Exterior corrosion has never been advanced enough to seriously affect the life of the structure. Interior compartments of caissons, which are constantly being flooded and pumped out to raise and lower the caissons when using the dry docks, show corrosion. Some of the compartments have been coated with "ballast tank coating", a thick crude oil, which is satisfactory when new but requires renewal annually.

"The best method of protection is by coating the interior with bitumastic compound. The composition of this material is of importance but is only secondary to the proper application of it. To apply this material successfully, steel plates must be cleaned by a covering of liquid preparation. This is followed with the bitumastic, which must be heated to apply it. The material must be applied when the walls are both clean and dry. Ventilation of the interior structure is necessary for the health and comfort of the men working in the compartments and also for the purpose of preventing condensation and sweating.

"The cost of this covering ranges from six to seven dollars per ton of structural steel involved. It is in general use for protecting interior compartments of ships. There are substitutes on the market, but no satisfactory ones that I know of. The interior compartments of the floating dry dock "Dewey" have been covered with this preparation. The floating dry dock at Erie Basin, South Brooklyn, with steel wing walls, has the interior walls coated with such compound, in which particular case it was a failure. The corrosion in the interior of this dock after about seven or eight years of use was very advanced. In the particular instance, I believe the failure was

due to doing the work improperly—that is, using the dock and wetting the walls before completing the coating.

"In steel ship-building generally, the steel plates, as you are aware, are first pickled, which consists in immersing the plates in a bath of dilute acid and then washing these off to remove the acid. The purpose of the pickling is to remove all mill scale. Interiors of hulls are painted with red lead and with white lead or zinc or given a coat of bitumastic compound, or in some cases some type of "ballast tank coating" of crude oil is used.

"Exterior of steel hulls is supposed to be painted, if possible, semi-annually, the painting being for two purposes: first, prevention of corrosion; second, prevention of fouling—that is, adherence of marine growth in the bottom. Generally two coats of paint are used: first, an anti-corrosive paint, and over this an anti-fouling paint. The anti-fouling paint used in the naval service deteriorates rapidly when exposed to the atmosphere, and it is only put on shortly before a hull is placed in the water. Extensive experiments have been made with a view to securing satisfactory paints. Various types of anti-corrosive and anti-fouling paints for ships' bottoms are on the market, the most satisfactory apparently being of European manufacture".

Mr. Louis J. Affelden.—"There is nothing unusual about the durability and preservation of steel in barges. If they are properly protected by paint, both inside and out, the metal will be preserved just as well as in any other steel structure. I would not attempt to state what this covering should be, although the specification for paint usually requires red lead and linseed oil".

Mr. J. A. L. Waddell.—"Asphalt makes a very good protection, as does also graphite if properly applied".

Mr. O. H. Ammann.—"There are many equally good paints on the market, but their efficiency depends very largely upon the manner of application. Unfortunately, incompetent labor is rather the rule than the exception. It is very important that the steel be thoroughly clean, free from dust and absolutely

dry, conditions which are not always observed; conscientious supervision is necessary to insure this''.

Mr. A. W. Buel.—''My experience with asphalt and graphite paints has not been as satisfactory as with red lead and oxide of iron. A few years ago it was impossible to get red lead more than about 68 percent pure. Now we get a red lead 98 percent pure. This makes a wonderful difference, because the old red lead contained thirty or more percent of litharge, which combined with the oil to form a soap. The new ''Dutch Boy'' red lead runs over 98 percent pure, is the finest known pigment except perhaps a lamp black, and does not contain sufficient litharge to chemically change the oil. My belief is this is the best paint up to date''.

Mr. H. D. Bush.—''Pipe which has been dipped in asphalt lasts a long time in the ground. The Allegheny City 60-inch (½-inch plate) riveted pipe was coated with Dr. Sabins' baked enamel, which was apparently a perfect protection. I noticed, however, that after this pipe had been tested in the field, pipe being filled with water under pressure, that when the water was let out there were thousands of little pin holes in this coating, which very quickly showed minute signs of rust''.

Mr. A. F. Robinson.—''As to preservative coatings in the ordinary sense, the investigations and reports of Committee D-1, on Preservative Coatings for Structural Materials, of the American Society for Testing Materials, are believed to embody the most complete and authoritative information on this subject''.

The Proceedings of the American Railway Engineering Association for 1915 describe paint tests of steel plates and on successive panels of a bridge over the Susquehanna River at Havre de Grace, which were commenced in 1906 and completed in 1914. The conclusions presented are: (1), six or eight years' protection may be obtained from commercial paints when properly applied in three field coats to carefully cleaned bridge surfaces (possibly excepting floor systems) and subjected to ordinary weather and train service conditions; (2) that (with a few exceptions) the serviceability of the coatings

increased with their thickness; (3) that test plate results give a valuable indication of service conditions.

The Angus Smith process, extensively used for the protection of water pipes from corrosion, consists essentially of baking the pipes coated with linseed oil and afterwards immersing them in tanks of hot coal-tar pitch.

That bituminous painting and dipping are useful preventives of rust when light is excluded from the surfaces covered with it.

In an article on "Powerful Influence of Basic Pigments in Protecting Metals from Corrosion", published in the Engineering Record, July 26, 1913, Mr. Henry A. Gardner, Assistant Director, Institute of Industrial Research, Washington, D. C., says that paint 82 to 88% red lead and 18 to 12% litharge with less than 0.5% total impurities, is highly inhibitive and more acceptable than any other for general painting purposes.

Ordinary Portland cement and furnace slags have been used with some success as pigments for metal paints. The development of basic lime compounds which takes place when moisture comes in contact with such pigments, if in sufficient concentration, protects metal from corrosion.

By-product tars from coke ovens, gas ovens and other sources make very good paints for metals exposed to acid fumes within buildings, but do not have the property of withstanding exterior exposure.

Iron oxide of good quality gives very fair results on metal surfaces. Black precipitate of oxide of iron is, on account of its highly basic nature, a pigment of great value in the manufacture of metal paints. In fineness and hiding power it is superior to all other forms of iron oxides.

Advanced practice for the preservation from corrosion of the largest bridge in the world is illustrated by the following extracts from the specifications for painting the 1800-foot span of the Quebec bridge, Mr. C. N. Monsarrat, chief engineer.

The paint shall be made of pigment, thoroughly mixed, in boiled linseed oil, without spirits of turpentine. The oil shall be pure and clear linseed oil, boiled with lead or manganese to a maximum specific gravity 0.939. The pigment shall be pure red lead with addition of lampblack not to exceed 4 ounces of lampblack to 30 pounds of red lead for the shop paint.

No dryer will be added to the paint unless authorized by the engineer, and shall in no case be more than 3 percent, with the exception of the paint used on materials before riveting. Dryer shall be made of linseed oil boiled with lead or manganese dissolved in spirits of turpentine.

All rolled metal work shall be kept under cover as far as practicable from the time it is rolled until it is painted and no material which has been punched or planed shall thereafter be exposed to the weather until it has been painted. All material arriving from the mills shall be unloaded without delay and protected from rust by being stored under cover or by the application of a coat of pure boiled linseed oil.

Before painting at the shop, all material shall be thoroughly cleaned of scale, rust, grease, dirt, chips and borings with steel scrapers and brushes or by other efficient method. Benzine shall also be used whenever required by the inspector for this purpose.

The paint shall contain as much pigment as possible, be kept well mixed before and during painting, and applied with brushes, and be well worked into all joints and surfaces. Wherever the paint runs or streaks, a fresh coat shall be applied. In riveted work the surfaces coming in contact shall each be painted before being bolted together and the paint must be dry before assembling. After the pieces are finished in the shop they shall be given one good coat of paint.

Machine finished surfaces, except faced ends of members which shall be painted, shall be coated with white lead and tallow before leaving the shops. All the painting before shipment specified above shall be done under cover and with metal dry and free from frost. The pieces must remain under cover until the paint is perfectly dry.

After the steel is erected, it shall be thoroughly cleaned and any parts where the paint has been scratched off or removed, shall be painted with red lead. The whole work shall then be given two additional coats.

F. W. Hibbs.—"In the first place, I should like to express my decided conviction that galvanizing is a thoroughly effective method of preventing corrosion. In ship-work it is the rule that all deck fittings that are not made of composition should be galvanized. Good galvanizing does not peel off; where the work is thoroughly and carefully done, there is an actual combination of the zinc and the iron or steel; the zinc cannot be removed alone, and the line of junction cannot be located. Peeling of the zinc occurs only when the iron surface is not absolutely clean, or when the piece to be galvanized, being of too massive proportions, is not heated through before or during the process, or when the zinc, being too cold, is deposited in too heavy a coat.

"As an example of the efficiency of galvanizing, I will

mention two torpedo boats, one of which was plated over her bottom with galvanized steel plates, the other with black steel. The first boat was built about three years before the second, and was docked about every six months; the bottom was always found to be in excellent condition, there being no rusting or corrosion except at the points of the rivets.

"The second was seriously corroded from the start; in fact she was docked as often as five times in one year for this reason. The case was of much more serious importance than in a larger vessel: in the first place, on account of the thinness of the plates, and in the second place, because there was the greatest difficulty in getting the paint to adhere in a sufficient body to protect the steel. It flaked off in large patches, due probably to vibration, internal heat, and the speed of the boat; and when it did not flake, the rust came through the paint. After numerous applications of paint, and repeated experiments, a sufficient body was formed to protect the steel fairly well; but it never did so well as the galvanized bottom.

"There can be no question of the efficacy of galvanizing the bottom of a large ship; but the expense is prohibitive, and with good attention, there is usually a sufficient thickness of metal to withstand the corrosion that ordinarily takes place, although it is frequently necessary to renew large portions of a ship's bottom plating on account of weakness due to corrosion".

Mr. C. E. Fowler.—"Were the writer in the position of some of the engineers connected with our great railroad systems he would undoubtedly favor galvanizing the overhead braces of bridges which are eaten away so rapidly, keeping them thoroughly painted as well".

Mr. F. R. Harris.—"On steel fastenings for wharf work, I have found that galvanizing for under-water work does not greatly retard corrosion".

Mr. A. W. Buel.—"The only work I have been connected with that was galvanized was for wireless telegraph towers, but we have given up galvanizing even them. I think that probably the reason that it was not successful was that the work was not carried out according to specifications, owing to the failure of the shop management to pay sufficient attention to details".

OTHER PRESERVATIVE PROCESSES.

Sherardizing consists of the formation of a zinc coating by heating the steel in a drum with powdered zinc or zinc vapor.

The Lohman Process for preventing corrosion of steel is claimed to weld a thin layer of zinc, lead, tin, antimony or other non-corrosible metal to the surface of the cleaned and sterilized surface of the steel. When applied to smoke jacks in round houses and when subjected to salt water action it is claimed that there were no signs of under-surface deterioration and that the Lohmanized steel lasted three or four times longer than galvanized material and then showed no signs of deterioration.

Mr. C. E. Fowler.—"The Barffing Process consists in forming on the surface of the metal a skin of magnetic oxide of iron, which does not further oxidize. The metal is heated to approximately one thousand degrees Fahrenheit, and while at this heat, superheated steam at the same temperature is admitted to the retort and the magnetic oxide at once begins to form on the surface. When the heat in the retort sets free the hydrogen in the steam, then the oxygen unites with the iron to form magnetic oxide.

"From a few hours to as high as twenty-five are required to complete the process—the thickness being from a hundreth to a tenth of an inch. There are various other processes to reach the same result: one of which is the Bertrand Process, which consists in coating the metal with a thin film of bronze and then heating the metal up to one thousand degrees Fahrenheit, the film of the metal which was deposited disappearing and the magnetic oxide forming on the surface.

"For small pieces, rods, or irregular shaped pieces of metal, this process is undoubtedly preferable for many uses to galvanizing".

Mr. George P. Lucas.—"Concerning the subject of corrosion of steel and iron in subway structures, I beg to say that no cases of corrosion have been brought to my attention to date. We have examined a great deal of steel that has been imbedded in concrete and then exposed in opening of structures

for connections, etc., but have found steel, when imbedded in concrete, in excellent condition; in fact so far as we could determine, in the same condition in which it was placed. We have never had any complaints regarding corrosion of steel and iron called to our attention by the Interborough in its repair or re-construction work in connection with the subway''.

Mr. F. R. Harris.—''On steel used for reinforcing concrete my observations, which in most cases were only general, have been that, under usual conditions, with a rich homogeneous mix, there is no corrosion or deterioration within the period observed. With the reinforced concrete submerged in salt water the same statement applies, except in such instances where the concrete deteriorated and disintegrated to such an extent as to expose the steel. On reinforced concrete slabs covering conduits or subways, where such subways carried steam pipes, the resulting high temperature, in spite of insulation, brought about the entire disintegration of the concrete, exposing the steel reinforcement to damp earth, the water causing its corrosion and destruction in a few years''.

Mr. A. W. Buel.—''As to cinder concrete, most of the experiments show that there is a great deal of danger from corrosion due to the sulphur contained in the cinders or ashes.

''The trouble with the pure cement covering is the difficulty in keeping it sufficiently wet to obtain perfect hydration, unless it is used in the shape of a mortar of concrete not less than 1½ inches thick. I think that there would be no better protection than a cement or mortar covering from 1½ to several inches thick, with wire fabric imbedded therein''.

Mr. F. C. Osborn.—''Referring to my own experience, I happened to be in Hamilton, Ohio, at the time of the taking down of the old suspension bridge. This was, I think, about 1898. I remember noticing the anchorage bars which were embedded solidly in concrete. I think I was told that the bridge had been put up about 1830, but, as the concrete was removed, the anchorage bars showed up with the paint apparently as fresh as when it was put on. There was apparently no depreciation whatever''.

Mr. Jonathan Jones.—''On recently breaking up some balusters and hand rail, about eight years old and exposed to

all weathers, in which slightly rusted steel rods were covered with not less than three inches of granolithic concrete, I found there had been no progression in the rusting.

"For some years past the City of Philadelphia has encased its steel girder bridges over railroads in concrete of not less than three inches thickness. Under favorable conditions and with proper maintenance, a painted steel bridge can be counted on for forty years of life. The absence of signs of deterioration to date in the concrete-protected bridges indicates a much greater expectation of life, with a freedom from expense for maintenance. Some approximate calculations indicate that in spite of the great initial cost, the concrete-protected bridge is cheaper after about sixty years. It has, moreover, greater margin for future increase of wheel loads and density of traffic".

Mr. F. W. Hibbs.—"Ballast tanks should always be coated with cement wash on the bare steel. Although the cement will sometimes flake off, it gives better protection than any paint. This is even more true of fresh-water tanks, as fresh water, and particularly distilled water, softens and permeates paint more than salt water.

"Where mixed cement or concrete is used, it should be applied to the bare steel without paint; and it is also advantageous to apply a coat of cement wash first".

Mr. J. A. L. Waddell.—"If pure cement covering can be made to stick, it ought to be very effective.

"As far as I know, stone concrete protection is most effective. It is better not to have the metal painted, but it should not be rusty.

"I believe cinder concrete in contact to be bad. The only excuse for using cinder concrete is that it is lighter than stone concrete. That means that it is porous, and will let the water in, consequently, the steel that it covers would not be protected".

Mr. A. D. Flinn.—"Water tanks on board United States' ships at Newport News were protected by two coats of cement wash that resisted considerable blows and was found in good condition after three years' service".

In a plate girder bridge carrying the main line tracks of the Erie Railroad across the Erie & Jersey Railroad at Guymard, N. Y., the main girders are entirely enclosed in concrete continuous with the solid floor concrete which fills all the spaces between the floor beams. The girders are supported at one end, on steel columns embedded in a continuous concrete pier wall.

DESIGN.

Mr. J. P. Snow.—"Many bridges show distress under service from wear or crushing when metal rests on metal. Deck plate girders fail sometimes by the cutting of rivets and rivet holes in the top flanges, due to insufficient contact bearing of rivets on the web. This action frequently develops in floor beams which rest on lower chords in the old-fashioned way. It is analogous to the working of members on pins which frequently happens in bridges of somewhat poor design. The remedy is, of course, ample metal. I have strengthened many cases of weakness of this sort.

"I have been acquainted with many cases of very bad intersections in riveted connections and of excessive computed bending moments on pins; but have never observed any distress from secondary strains in the former case or trouble from weakness of pins in the latter. Loop-welded eyes sometimes fail gradually by splitting or parting of the weld, and the webs of floor beams split and tear if weak in shearing strength at the ends. Supports for stringers on masonry, built up of plates and angles, frequently suffer from the crushing and crumpling of the bearing angles. This action is accelerated by the corrosion induced by the dirt, etc., which generally accumulate around bearings on masonry. The hammering effect of the trains is much greater if the reaction is rigid like a bearing on masonry than if it is part of the elastic framework.

"In the case of deck plate girders spaced so as to be nearly under the rails, the pounding effect of the wheels frequently bends down the girder flanges when the ties are somewhat softened by decay. A much better bridge results from setting the girders wider apart and using floor beams with rolled

beams for stringers under the rails, the ends of ties resting on the girders. This floor constitutes an admirable transverse bracing and also makes part of the top lateral bracing. No bottom laterals are needed''.

Mr. Aubrey Weymouth.—''No doubt the design has much to do with the life of a structure subject to vibration. I have recently examined a structure the strength of which was greatly decreased by the failure to properly brace the girders laterally. However, I think this matter is given more intelligent consideration now than it received twenty years ago''.

Mr. A. W. Buel.—''I had one or two examples several years ago of viaduct structures on curved alignment, which had practically nothing to resist the centrifugal force and wind except the track itself, but which had stood up all right for a long time''.

Mr. J. A. L. Waddell.—''If the designer be a true bridge expert in every sense of the word, his designs are not likely to be imperfect in any important particular, because today we know so much more about the science of design than we did a quarter of a century ago. The effect of imperfect design in existing structures should always receive serious consideration, and the more imperfect the design, the greater should be the reduction in the excess allowance in the working stress above that specified for new structures''.

BRIDGE STRESSES.

Mr. A. W. Buel.—''My observation is that the present tendency is to so design structures as to limit secondary stresses to the minimum. (See Report of Committee No. 15, A. R. E. Association on Impact)''.

Mr. J. A. L. Waddell.—''In my opinion, the old-fashioned methods of taking care of reverse stresses were more than sufficient, in fact, they were extravagant of material, and my present practice is to reduce their assumed effectiveness, and I make a distinction between reversion during the passage of a single loading and reversion caused by different loadings, on the theory that, in the latter case, the metal will have time to readjust itself before the reversion occurs.

"Secondary stresses in trusses should be figured, and if they are excessive, their effects should be met by inserting initial stresses of the opposite kind; but in most cases they are so small as to make it perfectly proper to ignore them.

"I do not see that camber stresses enter the problem.

"Initial stresses in structures should not be great enough to do any damage".

Mr. O. H. Ammann.—"Probably no influence is so detrimental to a bridge, and therefore shortens its life, as reversed stresses occurring under live load and not properly taken care of in working out the details. While the effect of reversal on the members themselves is probably not more harmful than the fatigue, it is this effect on the connections which is most detrimental, riveted connections being loosened, and in pin connections the members work themselves into the pins. The connection should therefore be made amply strong. The bearing pressure on the pins is, as a rule, assumed too high.

"Secondary stresses should be carefully investigated in any structure of unusual design so as to avoid excessive bending stresses which might decrease the life of the structure, particularly if such stresses are caused by the live load; but unless such stresses are excessive, no extra provision need be made, as the usual margin of safety fully covers these stresses".

OVERLOADING BRIDGES.

Mr. A. W. Buel.—"As to conditions of structures after long service, I once examined some wrought-iron lattice bridges, built many years previously by A. Lassig, which had been for some time under loads producing from twenty to twenty-two thousand pounds per square inch forty times a day, and which were in very good condition without any loss of rivets. I have also found pin-connected spans in which the computed fibre stress in the pins was over a hundred thousand pounds to the square inch, without any very noticeable evidence of overstress. Of course the explanation is that the assumptions of the computations were more severe than actually existed, and that the common theory is not very close to the actual results in such cases".

Mr. C. E. Fowler.—"When the writer was bridge engineer of the Hocking Valley Railroad, he had occasion to examine some sixty or more bridges for overloading, and developed a formula similar to the Launhardt formula for rational unit stress. Where it had been customary to allow as high as 20,000 pounds per square inch on certain members, this formula cut down the maximum unit stress to from fourteen to eighteen thousand pounds. This formula was carefully discussed in Vol. XLI of the Transactions of the American Society of Civil Engineers, page 203, and had the form of

$$S=\frac{E}{2}\left(1+\frac{\text{Min. } B}{\text{Max. } B}\right)$$

where E is the elastic limit and B the stress in the member".

Mr. J. A. L. Waddell.—"In considering old bridges that are overloaded, I generally assume that if the overload by the actual live loads, with proper allowances for impact, overstresses the material 50%, the bridge should probably be removed; but I make an exception of well-built plate girders by raising the limit to 60%. In case of very badly designed and loose-jointed structures, I would make the limit lower than 50%.

"Rapid loading is properly taken care of under the head of impact, and needs no further consideration".

Prof. Edgar Marburg considers that Cooper's E. 60 loading will be ample provision for designing railroad bridges.

IMPACT AND VIBRATION.

Mr. J. A. L. Waddell.—"Impact can be injurious to old bridges only by stressing the metal beyond proper limits.

"Bridge engineers today increase the live-load stresses by certain percentage to cover the greatest possible effects of impact, hence modern structures are not injured thereby.

"In a modern, well designed bridge which is rigid throughout, the injurious effects of vibration are practically *nil,* but in pin-connected structures designed and built many years ago, the vibration causes a cutting of pins in eyes, and, therefore, shortens the life of a bridge".

Mr. O. H. Ammann.—"There is no question that the impact due to rapidly moving live loads has a considerable influence upon the life of a bridge of short span, and must, therefore, be provided for. The way in which provision is made varies greatly. Impact formulas are generally not used in Europe, but instead smaller permissible unit stresses are used for shorter spans, while in this country impact formulas are the rule. Although extensive experiments have been made to determine the effect of impact, such as those conducted a few years ago by the American Railway Engineering Association (see Bulletin 125, 1910), the results are yet far from being conclusive.

"Impact in steel buildings may in general be disregarded as a factor in the life of the structure".

A large number of heavy printing presses transmitted vibrations 190 feet to a six-story building 49 feet wide with steel beams supported on brick side walls and interior columns and girders.

FATIGUE.

Mr. J. A. L. Waddell.—"In my opinion this factor is based upon a misconception, for I am firmly of the belief that there can be no deterioration of metal from fatigue unless the elastic limit be exceeded. The effect of impact, of course, should be duly included. As a properly designed bridge never has any of its metal stressed to anything like the elastic limit, the whole bugbear of fatigue of metal may be relegated into oblivion".

Mr. O. H. Ammann.—"Fatigue of steel may be disregarded as a factor in the life of steel structures such as buildings and bridges. The fatigue tests made by Wohler, in Germany, from the results of which certain formulas for safe unit stresses have been deduced, were made under conditions which are applicable to rapidly moving machine parts but not to steel structures. Those formulas have been mostly abandoned".

CRYSTALLIZATION.

That iron and steel are often found after severe service with a damaged crystalline structure, is undoubted, but it is

many times a moot question whether this condition was developed in service, or during the original manufacture of the metal.

Mr. J. A. L. Waddell.—"I believe that if the metal is not stressed up to the elastic limit, it will not crystallize. Of course, it may be crystallized in the rolling, but the tests made on it should determine that, and any crystallized metal should be rejected".

Mr. A. W. Buel.—"Crystallization was a very important matter in wrought iron, particularly in large squares, but I have not had any trouble with it in structural steel. I used to have a great deal of trouble with crystallization in the piston rods of steam hammers when I was shop superintendent, and found that they had a limited life. We got better service with high steel and also with cold-rolled steel than we did with low-carbon steel. I do not know that this has been very thoroughly investigated as regards structures".

Prof. Edgar Marburg.—"In your reference to tests of steel and iron after long service, permit me to take the liberty of cautioning you against saying anything that may tend to perpetuate the wholly erroneous impression that such a thing as the cold crystallization of iron and steel under service conditions exists.

"Only within a year such erroneous editorial reference was made in the Engineering Record to eye-bars that had been recently removed from the 315-foot span of the Fraser River cantilever bridge built in 1885, which were reported to have shown such deterioration. At my suggestion, the matter was further investigated by the editor, and some weeks later editorial acknowledgement of the error was made. I would suggest the desirability of availing yourself of this opportunity to come out strongly in objection to the so-called theory of cold crystallization, in support of which, in so far as I am aware, no scintilla of evidence exists".

Mr. C. E. Fowler.—"There has never come to the writer's attention an authenticated case of crystallization, although it may possibly take place in railway bridges where they carry heavy loads, and where the members are subject to many repetitions of stress".

Mr. A. W. Carpenter.—"In connection with crystallization, I would state that some was noted in the bending tests of the floor-beam hangers noted above, but much more marked crystallization was noted in the heads of eye-bars on some of the old West Shore Railroad bridges built in 1883 and removed in 1905 to provide for heavier loads. These bridges were of iron. Some of the eye-bar heads broke in removing the bars, showing completely crystallized structure. Very few such cases occurred".

An editorial note in the Engineering Record of December 13, 1913, states that after 27 years' service a deck cantilever bridge of 330 feet span did not show considerable deflection under the increasing live loads, but was replaced and "the tension members found so badly crystallized that in the opinion of the engineer in charge of the work it was a wonder that the bridge stood up".

Such a painfully indefinite statement, without any specific information, or any details or information concerning kind or quality of materials, type of truss, character of members, unit loads, or strength of material possesses no value nor significance other than a creditable record for a structure designed and fabricated under the disadvantages and imperfect art of long ago.

LOCOMOTIVE BLAST AND GAS.

Mr. J. A. L. Waddell.—"Locomotive blasts certainly are exceedingly injurious to metal, especially when the locomotive passes beneath the bridge and the smoke is held against the steel by the floor. The very best protection is required in such cases, and that protection, I think, is covering with a layer of concrete or grouting. The effect of locomotive fumes on overhead places can be taken care of all right by careful painting.

"Careful painting will prevent any bad effects from atmospheric action.

"If the air in the neighborhood of the structure is constantly contaminated, the best course to take is to renew the paint often".

Mr. Edward B. Guthrie.—"What I have to say is limited to bridges over railroad tracks and to the protection of the floor system and lower chords. In so far as the structures

erected by this Commission are concerned, we have had nothing to do with the maintenance thereof, and therefore I can only give you ideas as to results based on casual inspection and general information concerning same after we have built them.

"The first four structures were not protected and the clearance in some cases was but sixteen feet, in consequence of which two of them have been practically rebuilt, so far as floor system is concerned, and the others will have to undergo extensive repairs, if not reconstruction, though there may be salvage in the through trusses of the latter.

"From 1897 to 1900 a pine ceiling protection was used, but this proved ineffective mainly because the parts thereof could not be held in place by reason of corrosion of the nails. From 1900 to 1907 we tried protection by means of concrete on the floor beams and lower chords, but the low clearance gave opportunity for the blasts to cut off this protection and then the gases caused corrosion. From 1907 to the present time we have used concrete with asbestos wood, concrete with creosoted wood, and structural steel encased in concrete, which, so far, have given fair results but can be improved in some minor details, particularly the method of securing asbestos or creosoted wood to the concrete.

"We are preparing plans for a bridge with the floor system encased in concrete, with creosoted wood secured to the concrete by expansion bolts and the wood secured by copper nails.

"I believe something on the latter lines, or cast-iron plates as have been used elsewhere, will eventually be successful in protecting these bridges, and particularly where the clearances are slight. In a general way, I believe that with all such structures, no matter what the protection may have been, the results would have been better if greater care were taken in maintaining them and looking after them from year to year".

Mr. J. P. Snow.—"An interesting instance of the corroding influence of locomotive gases occurs in railroad cuts, especially ledge cuts, where the banks are nearly vertical. If these banks are 10 feet or more in height there will be a distinct difference observable in the rusting of tie plates and spikes, rail braces,

etc., within the limits of the cut and on the fill outside the cut. The banks retain the gas enough to cause this difference''.

Mr. A. W. Carpenter.—''In connection with the life of steel and iron highway bridges located over steam railway tracks in unfavorable locations, herewith is a partial record of the dates of construction and of renewal, in whole or in part, due to corrosion, for a number of actual cases.

''**Some typical cases of overhead highway bridges located over yard tracks, renewed in whole or part on account of corrosion of the parts below the floor.**

Case	Material	Year Built	Year Renewed*	Years in Service Before Renewal.*
No. 1	Wrought Iron	1886	1906-8	20-22
No. 2	Wrought Iron	1889	1911	22
No. 3	Wrought Iron	1892	1909	17
No. 4	Steel	1895	1906-B	11-13
No. 5	Steel	1902	1911	9
No. 6	Steel	1897	1915†	18†''

Mr. J. R. Worcester.—''There have been a number of instances in this vicinity of bridges over railroad tracks which have been practically destroyed by smoke, etc. I have no doubt that you are personally acquainted with all of these instances, and that I can add little to your information. To obviate the trouble, the city of Boston has, in some cases, protected steel-work with concrete, using heavy channel irons underneath floor-beams to protect the bottom flanges where they were afraid the concrete would not stick. The city of Cambridge has economized on this channel-iron guard by using wooden planks under the floor-beam flanges. These seem to serve the purpose and to last very well''.

Mr. A. W. Buel.—''For protection from direct blasts of locomotives when nearer than five or six feet, I believe that nothing has proved satisfactory except cast-iron shields, or other similar devices''.

Mr. George H. Tinker.—''After 28 years' service, the wrought-iron bottom-chord bars 17 feet above the rail in the

* Renewal of corroded parts, such as stringers and floor beams and reinforcing of other parts to make up for weakening due to corrosion.

† Closed to traffic on account of corrosion.

roof of a train shed had their sectional area reduced about 70 percent by locomotive blast. After 30 years' service the wrought-iron rods in a round-house roof were reduced 30 percent, and some of them showed crystallization. In Chicago, floor beams exposed to locomotive blasts lost ¼ inch of metal in two years.

"The Cottage Farm bridge over the tracks of the Boston & Albany Railroad, Boston, had a clearance of 15 feet above the rails. The bottom flanges of the floor beams were enclosed in lead plates on which were seated the skewbacks of terra-cotta floor-arch tiles. After 10 years' service, the locomotive blast destroyed the tiles and cut into the lead plates so much that both had to be renewed".

Overhead bridges with massive concrete protection from locomotive gases are illustrated by a short-span skew structure carrying four tracks of the Lake Shore & Michigan Southern Railway, near Vermillion, Ohio. The floor consists of 15-in. I-beams 12 in. apart, imbedded in a solid concrete slab 20 in. thick and supported on intermediate I-beam girders and steel columns inclosed in a solid, thin transverse pier of reinforced concrete. The upper surface of the floor is waterproofed and drained through numerous vertical scupper pipes, and the under surface is protected from locomotive blasts by cast-iron plates one inch thick and 30 in. wide, over the track centers.

All the steel work of the 631-ft. plate girder viaduct of the Chicago, Rock Island & Pacific Railway across the Chicago & Western Indiana Railroad tracks at Seventy-ninth Street, Chicago, has been incased in concrete applied with a cement gun to protect it from locomotive gases. The 1:3 concrete 1¾ in. thick was applied to galvanized wire mesh and was reinforced by ⅜-in. rods in the horizontal slabs connecting the floor-beams' lower flanges.

The highway bridge over the New York Central & Hudson River Railroad tracks at Genessee Street, Utica, has three pony lattice girders with their bottom chords protected from locomotive smoke by being imbedded in concrete with hollow spaces between the chord webs, and with wire reinforcement and anchorage outside the chords. This concrete is made continuous

with the concrete enclosing the floor beams. The concrete protecting the bottom flanges of the chord and floor-beam angles is shielded by wooden troughs secured to it by oak plugs.

The Genessee Street highway bridge over 15 tracks of the New York Central & Hudson River Railroad in Utica has a reinforced concrete slab floor on plate-girder floor beams, which with the lower chords of the riveted pony trusses are entirely enclosed in solid concrete protection against locomotive gases. The weight of the concrete is materially reduced by the insertion of permanent hollow sheet-metal cores. The bottom-flange concrete of the lower chords and floor beams is protected by wooden trough coverings secured to it with oak plugs.

A 146-ft. wrought-iron through-truss highway span built in 1878 carries Calvert Street, Baltimore, over the tracks of the Pennsylvania Railroad having a traffic of 500 locomotives daily. The paved buckle-plate floor 75 ft. wide is 26 feet above base of rail and is carried on I-beam stringers and plate-girder floor beams 3 ft. deep. The steel was much damaged by locomotive gases, and paint would not endure more than one season. In 1914 the floor beams were thoroughly cleaned by chemicals and sand blast and the surface protected by concrete applied with a cement gun at a contract price of $1.25 per square ft.

EFFECT OF SALT AND ACIDS.

Mr. J. B. Snow.—"Brine drippings from beef cars constitute another serious enemy to bridge metal and track fittings of all kinds".

Mr. A. F. Robinson.—"As to the action of salt or brine drippings from meat trains, we constantly had trouble from this source as long as we used open deck bridges. The use of ballast deck bridges has almost entirely done away with this trouble. The same might be said concerning special acids and liquids".

Mr. A. W. Buel.—"Regarding the action of salt, I am informed that the Lehigh Valley Railway has used with success American Ingot Metal Flakes on solid-floor bridges to protect the same from drip from refrigerator cars".

Mr. Jonathan Jones.—"Last year the Philadelphia and Reading Railway Company removed their truss bridge over Spring Garden Street, Philadelphia. This had a wrought-iron trough floor, painted and partly filled with coal-tar concrete. The up-troughs and part of the trough sides were in contact with the ballast. After some 20 years of service the wrought iron, including the rivet heads, was practically without evidence of deterioration".

Mr. J. A. L. Waddell.—"Salt drippings on steel certainly injure the metal, and the latter should be protected against them in the best possible way. Some railroad companies cover the floor with boards that serve as troughs to catch these drippings from refrigerator cars and lead them off of the structure without touching the metal work.

"I do not see how acids and liquids can well reach the metal-work except when contained in the air. Conditions that would allow of acids and liquids reaching the metal should be prohibited".

RIVETING.

Mr. A. W. Buel.—"As to shop work, and particularly riveting, almost all of the shops are now doing such excellent work that we have very little trouble. I have experienced no trouble in maintenance where the rivets and connections were properly designed so as to avoid eccentricities, tension on heads, etc. Where impact or rapidity of loading is great, as in stringer connections, the unit stresses for rivets should be at least 20 to 40 percent below the ordinary unit, or an excess of rivets should be used".

Mr. O. H. Ammann.—"Undoubtedly imperfect riveting shortens the life of a bridge. While the riveting in a first-class bridge shop is generally satisfactory, the field riveting is often imperfect on account of the necessity of using hand tools and often an inferior class of labor. The best remedy is competent and thorough inspection".

Mr. A. F. Robinson.—"We have had very little trouble from burned rivets and rivets tearing, as our rivets are taken out long before anything of this kind could occur".

Mr. J. A. L. Waddell.—"Imperfect riveting should not be allowed. If there be much of it in a structure, some of the rivets will eventually work loose, and if they are not replaced and otherwise taken care of, injury will result.

"No burned rivets should be allowed.

"In most cases loose rivets should be removed as soon as discovered, but in exceptional cases the removal may do more harm than good by the tendency to loosen neighboring rivets.

"Failure of rivets shows bad overstress and great damage to structure".

LIFE OF BRIDGES.

Perhaps the most wonderful example of the long life possible for iron or steel is that of the famous Delhi pillar, a solid iron shaft 12 inches in diameter and 24 feet long which has been standing for about 1600 years.

The wrought-iron bridge of the Pere Marquette Railroad over the Manistee River was subjected for 25 years to constantly increasing loads considerably larger than it was designed for, but an examination showed that all the bolted field connections, except those of the lateral bracing, were still in excellent condition. Two test pieces cut from members in service gave a minimum elastic limit of 41,230 lb., ultimate strength of 56,500 lb., elongation of 12% in 2 in., and reduction of area of 12.2%.

Prof. Edgar Marburg.—"It may, I think, be safely affirmed that the life of an iron or steel structure will be indefinitely prolonged (1) if it is properly designed, (2) if the safe load limits be not exceeded, and (3) if it is effectively protected against corrosion, it being, of course, understood that extraordinary contingencies that may wreck a structure are left out of account".

Mr. C. E. Fowler.—"The writer is strong in his belief that steel structures properly protected and not loaded beyond what they were designated for should last indefinitely, as is exemplified by the cast-iron structures in England and on the Continent which have been in use for upwards of one hundred years; however, this life for bridges will not be realized

in this country in county and city bridges until some more efficient method of supervision is found.

"The wearing out of iron and steel structures under load very seldom takes place, as they are usually taken down when they become too light to carry the increased loads, and are then available for re-erection at some other location where a light loading is all that the structure will have to carry.

"Over twenty-five years ago the writer took down an old railroad bridge which was too light for the traffic, and tests were made of a large number of the pieces to ascertain the condition of the material—all of which showed up good, except one top lateral, which was practically all crystallizing at the fracture. As this member, however, had no loading that could be any cause for crystallization, it undoubtedly was in that shape when it went into the structure."

Mr. H. D. Bush.—"We took down altogether 12 Miramichi Bridge spans of old Phoenix construction, with closed Phoenix columns. These spans had been up for 30 or more years over a tidal river where there was a great deal of fog. Notwithstanding the fact that these closed columns had been condemned by engineers because they could not be re-painted in the field, we found every one of these columns to be in perfect condition on the inside. If these spans had not been subjected to a much heavier traffic than that for which they were designed, they would apparently have been good, with occasional painting, for at least 100 years.

"The same kinds of spans as were formerly used on the line of the Pennsylvania Railroad between Baltimore and Philadelphia, over the Susquehanna River at Havre de Grace, are now in use in a highway bridge, and if properly cared for they will very likely continue in that use for another 50 years."

Mr. A. F. Robinson.—"For some eighteen years past we have been replacing old bridges that were anywhere from fifteen to thirty-five years old. So long as a bridge behaves itself properly under service, I do not trouble about replacing it unless the unit stresses are above 20,000 lb. net,—this to apply to the tension chords and the diagonals and floor system.

"We have taken out a good many bridges long before the live- and dead-load unit stresses reached 20,000 lb. As an

example, one bridge made up of six 150-foot deck-truss spans and located at the foot of a grade in both directions, where the track level was some 70 feet above water and where we could not maintain falsework, had to be replaced notwithstanding the fact that the unit stresses were scarcely above 16,000 lb. net. When I found that we could not make the spans hold their adjustment more than two months, or three at the outside, we arranged for replacing.

"In general, our bridges have more frequently been replaced because of faulty details or light floor system than they have for high unit stresses in the main members. As a rule, we have replaced our old spans when the unit stresses were anywhere from 50 to 90 percent above those for which the structure was designed."

Mr. George H. Tinker.—"The first bridges on the N. Y. C. & W. S. R. R. were built about 1882 and were of wrought iron. Renewal began in 1898 and continued to 1905. The reason for renewal was the continued increase in live load. The first bridges were built for 66-ton locomotives, the later for 136-ton, practically Cooper's E. 40. All bridges built since 1910 have been for E. 60. Several of the structures removed were in good condition and were sold to electric roads for re-erection.

"Some of those last renewed, at the time 136-ton locomotives were in use, were badly overloaded and showed excessive wear in details such as floor-beams, bearings, pin bearings, floor-beam hangers. Some skew spans showed the effect of unequal deflection by breaking of sway rods in the panels at ends of short spans. Deck structures were corroded by brine drippings from refrigerator cars. My experience indicates that iron and steel structures deteriorate rapidly when exposed to extreme corrosive influences, such as brine and acid fumes. When overloaded the poorly designed details fail. I know of no well designed structure which has failed under the normal conditions for which it was designed."

Mr. C. F. Loweth.—"The Chicago, Milwaukee & St. Paul Railway has a great many branch lines of short length over which the traffic is light and arbitrary where it is not necessary to employ heavy power or run at high speed. This makes it possible to retain in service bridges, which, for main lines

or even second-class lines would have been replaced long ago, by taking them out of service on the heavier lines and relocating them on lines where the traffic is relatively unimportant.

"We have also been able to take a great many spans out and use them for overhead highway bridges with little, or in some cases, no change.

"There are several factors determining the life of bridges, of which deterioration is perhaps the least important. I should say that the strength, as affected by the rapid and continued increase of rolling loads was the most cogent cause. Another and not inconsiderable factor is due to other improvements, such as, changes in alignment or revisions of grade, or changing from single to double track, all of which involve a changing of structures which may be otherwise all right. Some of the bridges included in the lists attached were in every respect satisfactory, but because of a change in alignment or grade, or the addition of second track, had to be removed.

"Another element affecting the life of steel bridges, as well as other structures, is the change in standards of construction. In some States they have clearance laws which are extreme and which prevent the re-use of old spans because of insufficient clearance. On some of our more important divisions we aim to resort to ballast floor construction wherever possible and this prevents the re-use of old spans which otherwise would be quite satisfactory. In a general way the above applies to other kinds of structures than bridges. The accompanying records show the life of our iron and steel bridges built previous to the year 1890. They cover, first, railroad bridges still in their original location; second, railroad spans taken out and re-used at other locations for railroad traffic, usually on secondary lines without change, or by doubling up in the case of deck girder spans on more important lines; third, railroad spans taken out and used in overhead highway bridges; fourth, railroad spans removed and scrapped.

"There are comparatively very few old iron and steel spans. The oldest metal spans we have a record of were delivered in 1873 at Bridges S-302 and S-338 and scrapped in 1907 after thirty-four years of service. The best record of a bridge still in service is thirty-nine years—built in 1876, used in its

first location twenty-six years and on a secondary line, thirteen years. The probabilities are that some of these spans will be good enough to round out at least fifty years of existence. The most important old bridge still in service is I-874 over the Missouri River at Kansas City. This bridge, built in 1887, is now being strengthened to carry engines as heavy as our L1 and L2 power.

"Not a great many spans have been scrapt, and those that have been usually consist of extremely long spans taken from important bridges, or are draw spans which are hard to work off in other locations. We are using many girders from old girder spans 30 ft. to 60 ft. in length for use in overhead highway spans, where a great deal of extra life should be obtainable after being used as railroad structures. There are a few overhead bridges, carrying highway traffic, built before 1890, still in service in their original locations. The most important one is Sangamon St. viaduct in Chicago, built 1881, thus having been in service 34 years and still being in pretty fair shape.

"No important span carrying railroad traffic has had to be removed because of deterioration, the use of heavier power accounting for most removals; and line changes, for practically all the remainder. Highway spans over our tracks, however, have had to be removed largely because of weakness due to deteriorated metal. Such cases are represented by the Sixth Street and First Avenue viaducts in Milwaukee, Wisconsin, and Milwaukee Avenue and Des Plaines Street viaducts in Chicago. The metal in all these bridges suffered much from the blasts from locomotives, and also became rusted enough through other causes to be affected in strength.

"We have had a few short railroad spans of less than 15 feet in length which had to be removed because of corrosion due to salt water drippings from refrigerator cars, and in one case had a 60-ft. through girder span of old design with stringers eaten so badly by brine as to require wooden stringers and eventually the replacement of the entire span, although this was not due entirely to the stringers. This corroding due to brine from refrigerator cars is much more rapid than corrosion due to rust, and on some lines is a matter of importance

in the life of the steel structures, particularly affecting stringers of bridges under the low rail on curves. Consideration is being given by the Master Car Builders to prevent brine leaking or dropping out, as it seriously affects track spikes and fittings, as well as bridges.

"Blast from locomotives very materially shortens the life of steel structures, particularly where the clearance is limited. At Halsted Street viaduct, Chicago, where the actual clear height above top of rail is less than 16 feet, we have an example of rapid deterioration. This viaduct was built in 1893, and the span over our tracks is now in such shape that it will probably have to be renewed in 1917, after 24 years of service. Since the bridge has been in service, practically all the I-beam stringers under roadways and sidewalks which are situated directly over the tracks have been renewed twice. The roadway stringers were last renewed two years ago. Sidewalk stringers are now eaten so badly that they are being replaced with wooden stringers. Beside these repairs, many of the floor-beam hangers have had to be replaced and several of the truss diagonals have had to be strengthened where eaten entirely through. Bottom chords are much reduced in section, but no repairs have yet been made. The deterioration is excessive here because switch engines stand for various periods of time directly under the span, and any paint is entirely removed by the locomotive blast, even before it has time to set."

Mr. A. W. Carpenter.—"The New York Central Company has many cases of wrought-iron bridges in railroad service approximating fifty years. These are usually, or perhaps exclusively, riveted deck lattice or plate girders originally carrying a portion of one track load and subsequently 'doubled up.' One case that I have in mind is that of a deck lattice-girder bridge originally carrying two tracks on three lines of M 29 girders, all of the same section, built in 1865. The three lines of girders were cut apart and grouped under one track at a later date and finally removed a year or two ago. These girders carried all the traffic on an important branch line. After final removal, sample pieces were cut from certain members, which we considered carried high stresses, and tensile

tests were made from these samples, the result of which I can furnish you if desired. The material was found to be very good.

"Two years ago we took down a large bridge, consisting of several spans with deck and through trusses and plate girders, built in 1876 to carry double track and subsequently remodeled for use as a single-track bridge in order to reduce the stresses. This also was on an important branch line. The material was wrought iron. Some time before the final reconstruction, some of the floor-beam hangers, consisting of loop-rods of round sections, were removed and subjected to full-sized tensile and bending tests, the result of which can be furnished if desired. After the removal of the bridge, full-sized tests were made on some of the eye-bars.

"Some years ago in providing instructions for estimating the cost of maintaining and renewing bridge structures, I prepared the data shown below, which are based partly on records of cost and service and partly on judgment (or lack of the same)."

Data for Estimating Maintenance Costs of Structures:

Renewals of New Steel Structures.

Railway bridges over highways and streams,—conditions favorable to non-corrosion ....40 years.

Railway bridges over steam railway tracks, or in yards, or in deleterious atmospheres—conditions favorable to corrosion....................25 years.

Highway bridges over tracks or steam railways in yard limits or other *unfavorable* locations—metal protected by paint only....................20 years.

Same as 3, metal protected by wooden sheathing, or highway bridges over steam railway tracks at high speed points and protected by paint only....................30 years.

Highway bridges over steam railway tracks, metal protected by concrete, or highway bridges over streams, highways or other favorable locations ....40 years.

Compute renewals on basis of 4% interest compounded annually, figuring capital required for renewal in periods given. In figuring costs of renewal, the old steel in structures may be credited at one-fifth the value of new steel fabricated, but not erected.

Painting.

For bridges of classes Nos. 1, 4 (covering bridges protected by wooden sheathing only) and 5 (except portions protected by concrete), as given above, estimate cleaning and painting one coat every 4 years at cost per ton of........ $1.00

For highway bridges the cost of removing the floor covering to paint parts beneath must be considered.

For bridges of class No. 4 (except bridges protected by wooden sheathing), estimate cleaning and painting once in 3 years, one coat, at a cost per ton of........ $1.50

For bridges of classes Nos. 2 and 3, estimate cleaning and painting one coat every two years, at a cost per ton of—

For class No. 2........ $1.00

For class No. 3........ $1.50

Compute, on basis of 5% simple interest, capital to provide painting as per above.

In a paper presented to the New York Railroad Club, May 21, 1915, Mr. Gustav Lindenthal stated that 100-pound rails should, on curves and tangents, probably have an average life of 100,000,000 tons, that a wheel tonnage of 375,000,000 is reported for exceptionally good 100-pound rails lasting 12 years with prevailing passenger traffic on the New York Central Railroad.

In a discussion of this paper, Mr. James O. Osgood, Chief Engineer, Jersey Central Railroad, stated that on sharp curves under a very heavy tonnage of long coal trains, 90-pound rails lasted from 6 to 18 months.

The life of structures may be materially prolonged by partial rebuilding or extensive replacement or reinforcement. The single-track wrought-iron viaduct of the Chicago & West Michigan Railway across the Manistee River consisted chiefly of about 60-, 30-, and 45-ft. deck plate girder spans on 75-ft. towers. It was built in 1888 for Cooper's E-25 loading, and in 1913 was reinforced to carry E-50 loads by transforming the spans into Warren girders by the addition of lower chords and web members and utilizing the old girders for top chords, the towers being strengthened by the addition of new batter posts with adjustable pier bearings in the transverse planes of the old tower bents. The new steel weighed 455 tons and cost $43,900.

Mr. George A. Orrok.—"In nine cases out of ten the steel structure will be superseded by another before it has been seriously weakened by corrosion, electrolysis or other physical factors."

Mr. H. R. Leonard.—"I know of no well-designed structure dying from other than neglect and overload."

BRIDGE ACCIDENTS.

Buildings, towers and other steel frameworks are much more subject to serious accidents that may reduce or terminate their life than are other steel and iron structures, and of these, bridges are much the most endangered, both on account of their elaborate construction and because of the severe and often unexpected service conditions.

Much the largest proportion of bridge accidents are due to derailments of trains, and to floods destroying their substructures or even directly injuring the superstructures. The danger from floods may be materially reduced by sufficient preliminary investigations of the river's regimen, and the provision of sufficient waterway, clearance and proper foundations. Danger from derailment can also be minimized by special care in the maintenance of track and grades, by operating restrictions, by the design of floor systems adequate to resist derailments without necessarily being destroyed, and by the provision in some cases of massive protection girders which have been furnished on some important spans to receive the impact of possible derailed cars and protect the truss members from injury.

There is occasional serious accidental damage to bridges during erecting, which is generally inexcusable except when due to violent storm, flood, or ice movement, and even then may sometimes be avoided by proper anticipation of probable conditions.

The most terrible of erection disasters was the total sudden destruction of the great Quebec Bridge a few years ago, when a completed anchor span and nearly half of the 1800-foot channel span collapsed, causing the instant death of about 75 workmen and the total loss of the structure. The accident was due to the failure of a compression bottom-chord member

under a static erection stress less than the computed working stress of the finished structure, and was still more deplorable in that this vital member had ample steel, of good quality, to safely endure this stress had its potential strength been fully developed, and because it was shown in the investigation that the same member had been reported in a crippled condition days before the disaster.

The 110-ft. pin-connected through single-track Pratt truss span of the Wabash R. R. bridge over the Wabash River at Attica, Ind., was wrecked April 5, 1914 by a slow-moving express train. It was built in 1892 at the Pencoyd bridge shops and the materials and workmanship were good. A few hours previous to the collapse the end post of the span had been struck by a derailed freight car and badly broken and displaced, so that it completely failed under the stress from a 421,000-lb. locomotive, which would normally have developed a unit compression of only about 7700 lb. in this member. Nearly all compression members failed near pin connections and nearly all tension members remained effective. The collapse of this span caused the failure of the adjacent 154-ft. span.

A very light riveted pony-truss highway bridge, only 39-ft. span, across the Hayes drainage ditch, in Tuscola Township, Douglas Co., Ill., failed in 1914 under the static load of an 18-ton motor tractor which tipped up the loose floor planks, badly twisted the I-beam and channel stringer webs and bent the members of one truss. The load developed computed unit stresses of 49,000 lb. in the stringers.

A 53-ft. deck girder span of the Cincinnati, Bluffton & Chicago R. R. across the Wabash River at Bluffton, Ind., collapsed under a train May 22, 1914, without having shown signs of previous weakness. The ends of the girder rested on piles supposed to be driven to bed rock. It was believed that the earth had been washed away from around the piles, allowing them to collapse.

Mr. F. C. Osborn.—"A suburban electric railway bridge at Tinkers Creek, Ohio, failed some years ago for two reasons. It was designed, as I recall it, for a suburban electric-railway car, but the railroad company began to use it for hauling fully loaded steam-railway cars. In addition to this overload, it

was found that in a Sub-Pratt truss two members of the same length, but of different dimensions, happened to be transposed, the light member being placed where the heavier one should have been used, and vice versa.

"The Tay bridge failure, as I remember, was caused by an unusually severe wind storm."

The Pratt highway bridge, Kansas City, has two concrete and asphalt roadways and a 30-ft. wooden-deck electric-railway floor in the center. About 530 ft. of the creosoted decking was burned in June, 1914, by a 13-hour fire, which expanded the viaduct members so much that the 5x3½x⅜-in. stiffener angles at the expansion joints were sheared at the roots in eight places, many rivets were sheared and 90% of the 20-in. I-beam stringers were buckled. The stringers were straightened in the field and the deck replaced by a reinforced concrete floor.

During the construction of the Manhattan Bridge, New York, the roadway wood caught fire and the heat caused the stringers to buckle so badly that some of them had to be replaced.

During the construction of the 1600-foot suspended span of the Williamsburg Bridge, New York, the hot fire caused by burning timber and oil on the scaffold on top of the New York tower caused the destruction of the falsework suspension span, and injured the permanent main cables so much that many wires in them had to be replaced. Some of the riveted members were also seriously injured."

COMPARATIVE LIFE OF IRON AND STEEL.

Mr. Jonathan Jones.—"In connection with the question of sheeting for mill buildings, it is well known that wrought iron, costing possibly one cent a pound more than steel, is a wise investment because of the considerably longer life."

Mr. John Carlin.—"I examined some ⅛-inch iron plates that were in a very good condition after being used for 50 years in tanks in a white-lead plant. Some of them have since been used for 5 years in a smoke stack."

Mr. Jonathan Jones.—"There are two bridges on Broad Street, Philadelphia, over the Reading Railway, one built in

1870 and the other in 1893; the former, built of wrought iron, is to date giving better service and requiring less attention than the latter, built of steel, which has required partial renewal.''

Mr. J. R. Worcester.—''The great difference in permanence between steel and iron is illustrated in a bridge over the Merrimac River at Tyngsboro, Mass., where the old wrought-iron portions, which are fully forty years old, are in a better state of preservation than certain additions of steel which were attached probably twenty years later. The wrought-iron structures, after they begin to rust, seem to be protected by the rust coating, so that corrosion does not proceed very rapidly underneath; whereas, steel seems to go from bad to worse after the corrosion begins.''

Mr. J. P. Snow.—''I once had a remarkable exhibition of this difference in some signal bridges which were built at the time when wrought iron was going out and steel coming in, in the fabrication of bridges. Stock material was allowed to be used in these signal bridges, with the result that most of the gusset plates were iron while the angles riveted to them were steel. Some years after they were erected, one was removed; and while many horizontal angles in the lower bracing were wasted to lacework, the plates were unharmed. I tested many of these pieces by nicking and breaking and invariably found the sound part iron and the wasted parts steel.''

Mr. J. P. Roe.—''Light sections of sheet iron last five times as long as the same sections of steel.''

Farmers' Bulletin 239, U. S. Department of Agriculture, states that mild-steel wire rusts about three times as fast as iron wire, and that it rusts in proportion to the manganese contained in it.

MISCELLANEOUS.

Mr. A. W. Buel.—''If I were asked to say what I consider the most important feature affecting the life of steel structures, I would say that it would be found in the matter of shop and field inspection. In field inspection I mean particularly the inspection of erection, because much punishment is suffered by careless erection. This is intensified by bad fits and lack of

clearance due to lax shop inspection. These points, together with lack of attention to details of shop work, proper cleaning before painting, etc., are in my opinion most important.''

Mr. J. R. Worcester.—''As I think over the old structures within my knowledge, I am forcibly struck with the importance of environment. I have known at least one highway bridge—located only a few feet above extreme high-water, over a tidal estuary exposed to a free sweep of water some miles in extent—which became so corroded as to be dangerous in about ten years.

''On the other hand, there are bridges in service today over rivers in Massachusetts which were built about forty years ago, and which show no appreciable deterioration from rust. Of course, there is this other element which comes into play to account for this difference, and that is that the older bridges are of wrought iron; while the one which I have referred to as lasting only about ten years, was probably of steel.''

Mr. J. A. L. Waddell.—''Failure of expansion devices is quite common in old bridges and should be avoided in new designs, mainly by using segmental rollers.

''Regular inspection and systematic maintenance are necessities, and the better they are attended to, the longer will be the life of the bridge.

''I do not believe that you will ever find any material deterioration of the strength of metal from actual use unless the structure has been loaded continuously up to near the breaking point.

''In modern bridges there are no adjustments allowed, but in old structures, with adjustable members, there is danger from overtightening the counters and other screw members.''

Mr. O. H. Ammann.—''Present mill processes so far as they affect the uniformity of the material, and therefore the life of a structure, are far from being perfect. Probably the worst feature is the present practice of casting the ingots in such a way as to cause piping and segregation, and very often not sufficient discard is made from the ingots so cast. The improvements suggested by Mr. G. Lindenthal in this direction for the manufacture of rails (see his paper on ''Quality of Good Steel Rails'', New York Railroad Club meeting of May 21, 1915) apply to a large extent also to structural steel.

"It is difficult to say just how far the fabrication processes influence the life of a structure. Undoubtedly the improved methods of fabrication, such as reaming and drilling, instead of punching to full size, reaming of holes to full size after assembling of connecting parts, assembling of trusses in the shop for reaming connections, improved methods and tools for riveting, will tend to increase the life of bridges and other structures.

"Loose pins are certainly detrimental to a structure, particularly, as already remarked, if the members are subject to stress reversal; since the member, in constantly working on the pin, gradually cuts into the pin, thereby increasing the looseness of the connection.

"Failure of expansion devices may affect the life of a structure by introducing certain additional stresses. Not too much stress can be laid upon the necessity of making such devices free from friction. Expansion rollers are still very often made too small in diameter."

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Paper No. 108

ALLOY STEELS IN BRIDGEWORK.

By

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According to the original request of the "Committee on Papers" of the International Engineering Congress, this communication should have covered the use of alloy steels in all kinds of engineering work; but the author being of the opinion that no engineer should address such an important assemblage as this except upon matters whereon he is an authority, and as more than thirty out of the forty years of his practice have been devoted to the specialty of bridge engineering, he requested the Committee to permit him to confine his remarks to the use of alloy steels in bridgework; and the request was granted.

The metals used at all generally in that branch of construction, enumerated in the order of their chronological precedence, are as follows:

Cast Iron,
Wrought Iron,
Medium Steel,
Manganese Steel,
Nickel Steel.

A century or more ago metal bridges were built entirely of cast iron, usually in the form of arches; then wrought iron, employed either alone or in combination with timber, came into vogue; then some thirty years ago medium steel began to be used instead of wrought iron, requiring six or seven years to supplant it entirely; and it was only a dozen years since that the alloy, nickel steel, began to be talked of seriously for bridge building.

Before the days of medium steel, though, a few large bridges were constructed of special steels, notably, in America, the Eads Bridge of St. Louis, Mo., and the Chicago and Alton Railway bridge at Glasgow, Mo. The latter was of Hay Steel; and although the author worked for a short time in a subordinate capacity on the structure, he has forgotten the composition and the characteristics of the metal, except, perhaps, that it was rather high in carbon. Be this as it may, the matter is of no special importance; because that make of steel, as far as the author knows, was never again used in any important bridge.

The term "manganese steel" for bridge work is somewhat in the nature of a misnomer; for all bridge steel has to contain a certain amount of manganese (generally from 0.5 to 0.8 per cent) in order to make it workable in the mills and otherwise satisfactory; but when a bridge engineer uses the term, he means a steel very high in manganese, and, consequently, exceptionally hard. Such a steel or alloy is employed for rail-locks and for those parts of the operating machinery of movable spans where great resistance to both abrasion and shock is the principal desideratum.

Chrome steel, an alloy of chromium with steel, the author has heard of as being used for this purpose, but not at all generally on account of its high price.

Chrome-nickel steel has also been employed somewhat for special castings in bridge machinery, but its principal use is for the manufacture of aeroplanes, automobiles, transmission lines, and gearing.

In most cases it is necessary to submit these various alloys to heat treatment in order to increase materially their hardness, elastic limit, and ultimate strength. The price for castings or forgings of such alloys generally varies from 8 to 12 cents per pound, according to the amount of shop-tooling required. Such prices, of course, are prohibitory for bridge-work, excepting only for small but important parts of operating machinery and for details requiring great resistance to abrasion.

Some manufacturers claim to be able to produce alloy steels having elastic limits as high as 250,000 lbs. per square inch; but it is impracticable to shop-tool them when the elastic limit exceeds 150,000 lbs. or when the ultimate strength is greater than

200,000 lbs. Such metal might possibly be required for bridge pins and their bearings in order to meet certain extreme conditions; but the probability of such requirement is exceedingly remote. Moreover, bridge engineers, as a rule, are loath to concentrate great stresses on members of very small cross-section because of the proportionately great effect thereon of any undiscovered small cracks or flaws which may exist in the metal.

While nickel steel was talked of for bridge work in both Europe and America prior to 1902, it was not employed therefor. In that year and in 1903 the well known consulting engineer and bridge specialist, Mr. Gustav Lindenthal, started some experiments upon the use of nickel steel for the eye-bars of the Blackwell's Island Bridge at New York City, reporting favorably thereon. Later, after trying hard to avoid its employment, the city authorities decided to adopt the alloy for the said eye-bars; and the bridge was constructed accordingly. This was the first actual use of nickel steel in bridgework.

In 1903, before the city authorities just mentioned came to their decision, the author inaugurated an exhaustive series of experiments and investigations upon the subject of the suitability of nickel steel for bridge building in general and its economics therein. In spite of many trials and tribulations, and in the face of strong opposition and great discouragement, he succeeded, after more than three years, in completing most of the work which, at the beginning of his undertaking, he had laid out to do; and it required some three months more to digest the results and to prepare a report for his principals, who were the International Nickel Company. That corporation financed the entire undertaking, spending altogether upon it nearly $50,000. As soon as the report was completed, the author devoted several months to the preparation of a monograph upon the subject, and presented it to the American Society of Civil Engineers. After taking considerable time for deliberation, the Publication Committee of that Society rejected the paper without giving any reason therefor. Fellow members of the Society especially interested in bridgework urged a reconsideration of the matter, which caused the Committee to reverse its original decision, provided that the author would agree to cut down materially the volume of the original memoir. The reduction was

accomplished by omitting some of the records of tests and all the diagrams and text relating to the economic study of bridges built wholly of nickel steel, retaining only those concerning structures built of mixed nickel steel and carbon steel. The original (rejected) memoir containing all records and diagrams is on file in the Society's library, where it can be consulted by anyone interested in this subject. The paper in due time appeared in the Society's ''Proceedings'' and was discussed by thirty or more engineers, both American and European; and finally it was published with the discussions in the 1909 ''Transactions'' of the American Society of Civil Engineers. Later the memoir was awarded the Norman medal, as being the best paper presented in that year.

The entire investigation proved (at least to the author's satisfaction, as well as to that of a large majority of the engineers who entered into the discussion) that nickel steel is in every way a suitable metal for the manufacture of bridge superstructures, being just as reliable as carbon steel and from 50 to 70 per cent stronger. The correctness of this statement is proved by the fact that the alloy was used later not only in the Blackwell's Island Bridge, before referred to, but also in the Manhattan Suspension Bridge at New York and in the Free Bridge at St. Louis. The last mentioned structure was designed and engineered by Henry W. Hodge, Esq., one of America's most noted bridge specialists. Again, the new Quebec bridge, which will contain the longest span in the world, viz., 1,800 feet, is to be partially constructed of nickel steel.

The author found by his experiments and investigations that it is perfectly feasible to produce commercially an eminently satisfactory nickel steel for bridgework, having a minimum elastic limit of 60,000 lbs. per square inch, a minimum ultimate tensile strength exceeding 100,000 lbs. per square inch, and an elongation in eight inches of fifteen (15) per cent. The actual extra cost per pound for this metal, delivered at bridge site, as compared with ordinary carbon bridge-steel, he figured should not be more than 1.5 cents.

Unfortunately, however, the steel makers and bridge manufacturers, being opposed, naturally, for pecuniary reasons to fun-

damental innovations in their business, have not responded to the call of the bridge engineers for a nickel steel of great strength at a moderate price, preferring to continue without interruption the production and manufacture into bridges of the cheaper carbon steel to which they are accustomed. For years they have refused to guarantee for nickel steel an elastic limit of more than 50,000 pounds per square inch, and they have asked therefor an excess pound price of from 1.5 to 2.0 cents. Mr. Hodge paid 1.65 cents per pound extra for his nickel steel in the St. Louis bridge; and the price named for the alloy in the new Quebec bridge was so high that it was found economical to use it only for the truss members of the suspended span and the cantilever arms. It is true that by adopting carbon steel instead of nickel steel for the anchor arms of a cantilever bridge, the weight of those arms is increased, and, in consequence, the stresses on the anchorage metal and the uplifts on the anchor piers are reduced, but these results could probably be obtained more economically in some other manner—for instance, by adopting a ballasted floor for the tracks on the anchor arms only.

In the case of his proposed bridge across the Entrance Channel to the Harbor of Havana, Cuba, the author, by great effort, succeeded in persuading the Carnegie Steel Company and the American Bridge Company to agree to furnish him with nickel steel having an elastic limit of 55,000 lbs.; but the extra pound price demanded for the manufactured metal was 2.5 cents. With these figures it was an exact stand-off between nickel steel and carbon steel for both the suspended span and the main structure as a whole; but as the said suspended span will have to be built on barges, floated to site, and raised by wire ropes to final position, the author concluded to adopt the alloy for that portion of the superstructure. He found also that it would involve a trifling economy to use it in the cantilever arms, but not in the anchor arms; hence he has decided to follow the same course as the designers of the new Quebec bridge did in relation to their great structure.

The compositions of the various classes of nickel steel for bridge work recommended by the author, in view of the results of his experiments, were as given in the following table:

Ingredients	Percentages Rivet Steel	Plate-and-Shape Steel	Eye-bar Steel
Nickel	3.50 (3.25 to 3.75)	3.50 (3.25 to 3.75)	4.25 (4.0 to 4.5)
Carbon	0.15 (0.12 to 0.18)	0.38 (0.34 to 0.42)	0.45 (0.4 to 0.5)
Phosphorus	0.03 Max.	0.03 Max.	0.03 Max.
Sulphur	0.04 Max.	0.04 Max.	0.04 Max.
Silicon	0.04 Max.	0.04 Max.	0.04 Max.
Manganese	0.60 (0.55 to 0.65)	0.70 (0.65 to 0.75)	0.80 (0.75 to 0.85)

The rivet steel specified is as high in carbon as it is practicable to go, in view of the fact that rivets must not be too hard to cut out when badly driven.

The carbon percentage in the plate-and-shape steel is as high as will permit of the metal being worked satisfactorily in the shops.

The percentage of nickel in both the rivet steel and the plate-and-shape steel is as high as considerations of both economy and workability allow.

As the phosphorus, sulphur, and silicon are in the nature of impurities, their percentages are kept as low as is consistent with economy in smelting; because it is expensive to reduce the said impurities below the figures shown—in fact, even these have raised objections among some steel makers.

In eye-bar steel it is permissible to make the metal harder than in plate-and-shape steel, because the shop-tooling on eye-bars is small in amount and of simple character; hence the percentages of both nickel and carbon, adopted above therefor, are quite high. The extreme limit specified for the nickel, viz., 4.5 per cent, causes the alloy to approach the brittle zone, which begins at some as yet undetermined figure between 4.25 and 5.00 per cent and ends at 20 per cent. This brittle zone was discovered by three English metallurgists, Messrs. Carpenter, Hadfield, and Longmuir, and was described by them in November, 1905, in a paper read before the Institute of Mechanical Engineers of England. It is claimed, however, by some American experimenters that the use of more than five (5) per cent of nickel does not, of necessity, make the steel brittle; hence it is likely that the percentage of carbon has some influence on the brittle zone of alloy steels containing more than the said five (5) per cent of nickel. Be this as it may, though, an engineer should test carefully for

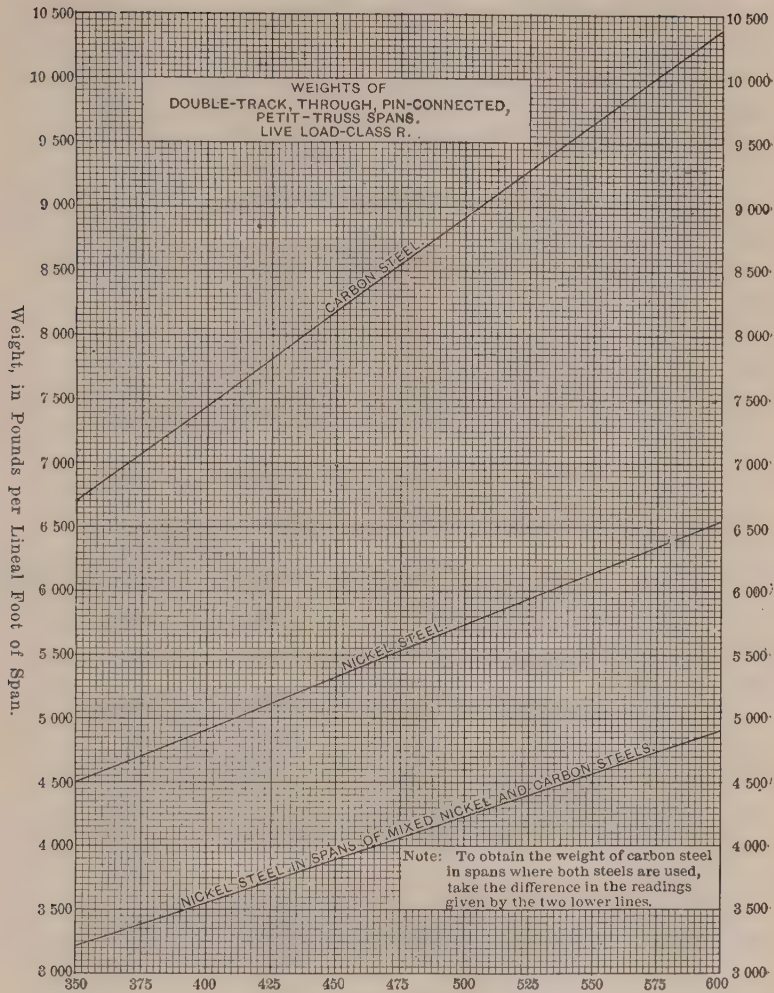


Fig. 1.

brittleness his eye-bar steel, if he employs in its manufacture nickel in any greater percentage than 4.25.

As shown in the preceding table, the amount of manganese in nickel steel is graded to meet the requirements of both strength and hardness. It varies from 0.6 to 0.8 per cent.

In the memoir, "Nickel Steel for Bridges", are given twelve

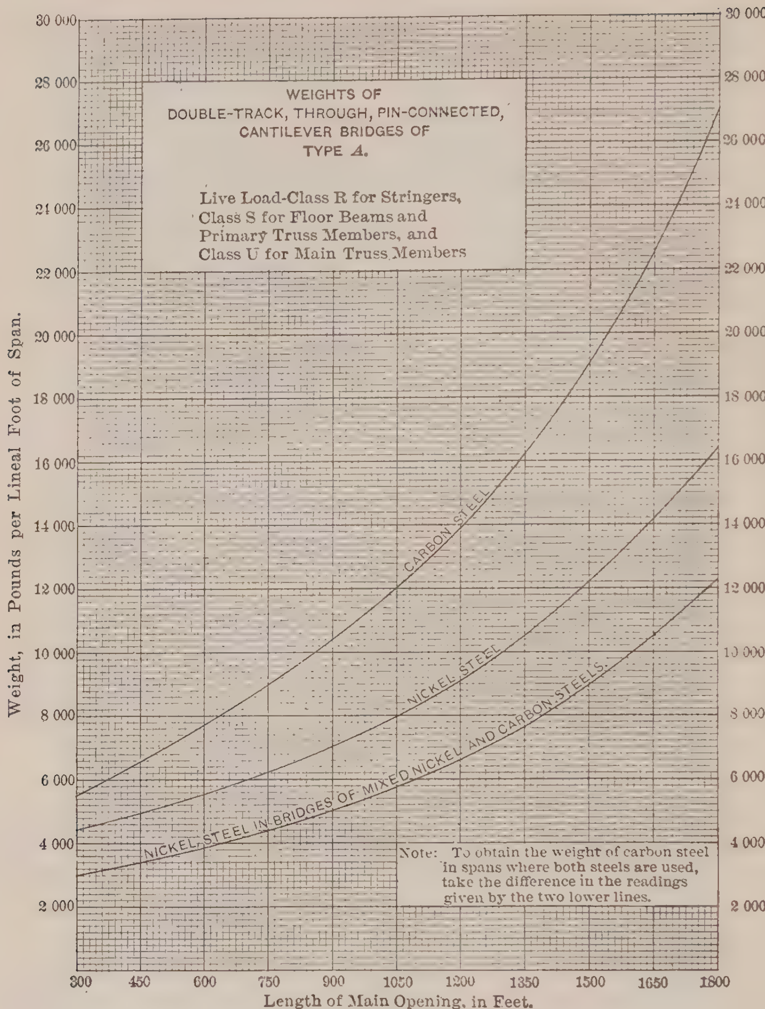


Fig. 2.

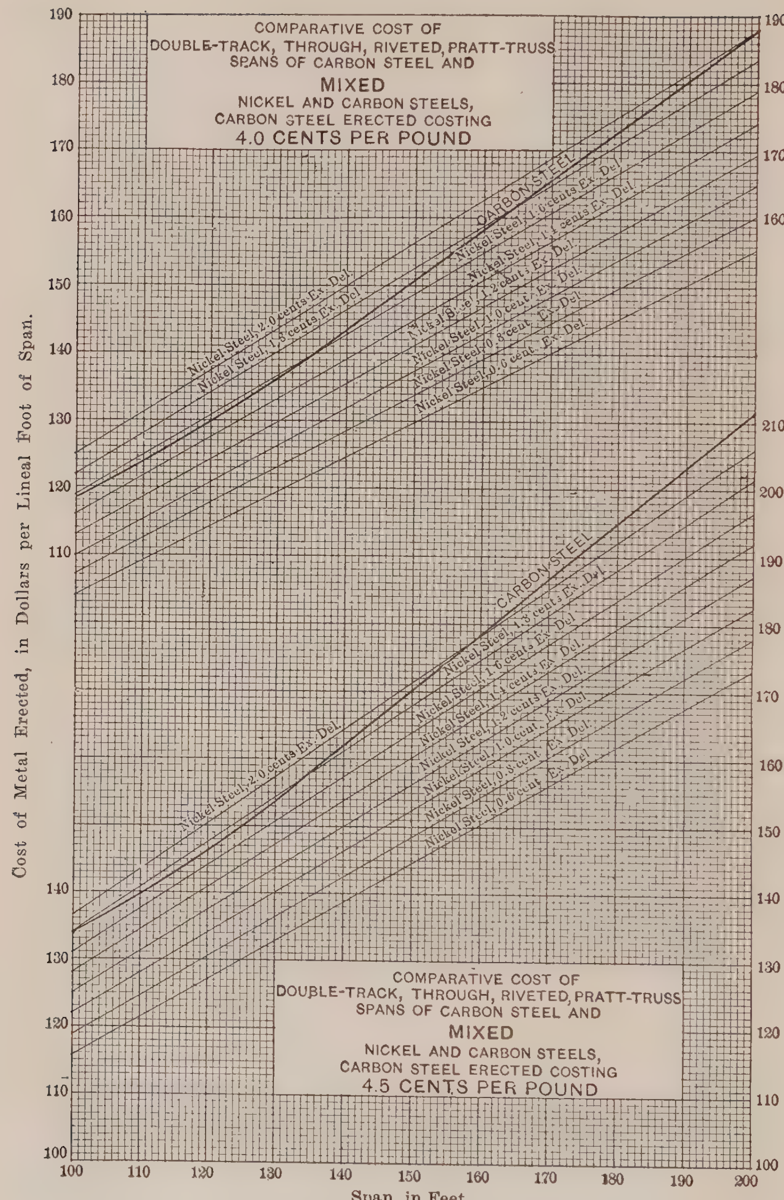
diagrams of weights of metal per lineal foot of span, covering all lengths from 20 feet for plate girders up to 1,800 feet for cantilever main openings. In Figs. 1 and 2 of this memoir are reproduced two of the most interesting and useful of those diagrams, covering double-track, through, pin-connected, Petit-truss spans and double-track, through, pin-connected, cantilever bridges of

the most usual type. The weights given are for structures built both wholly and partially of nickel steel, and for those composed entirely of carbon steel.

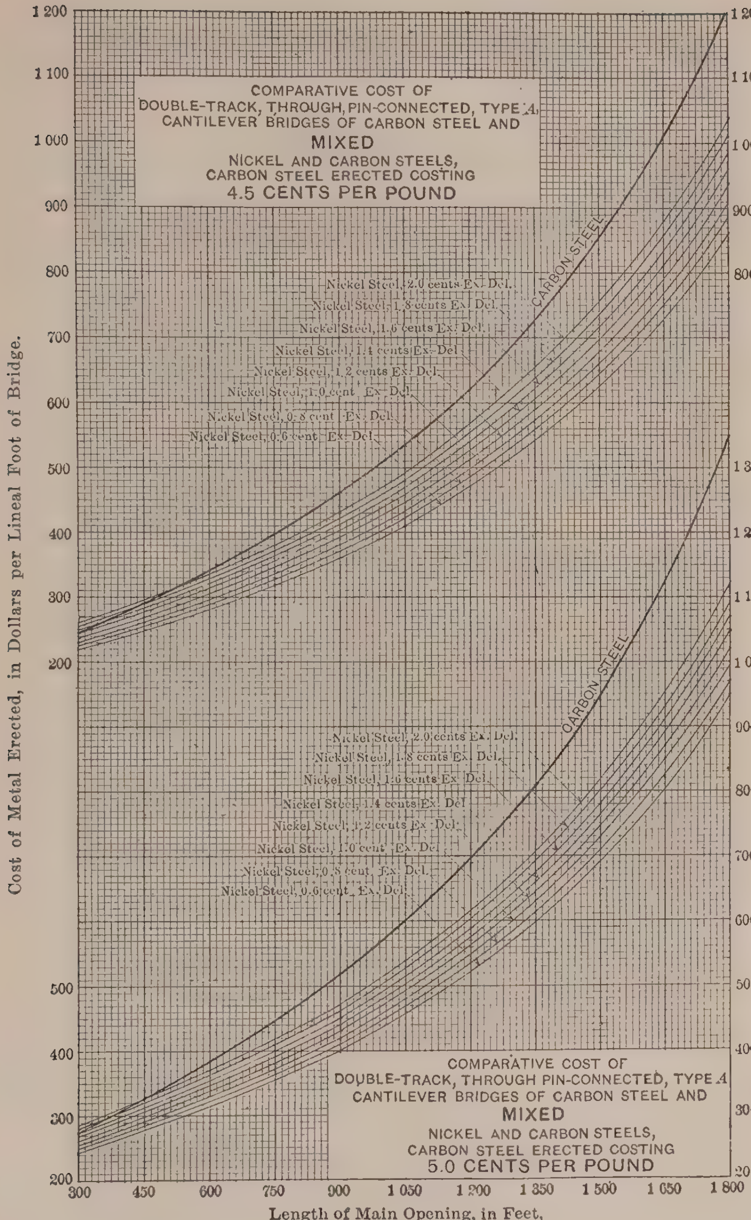
Following the twelve weight-records in that memoir come some fifty economic diagrams, which show for all span-lengths up to the before-mentioned limit, for both riveted and pin-connected structures, for all possible conditions of the metal market, and for all probable variations in pound prices between the manufactured nickel and carbon steels, the comparative costs of bridge superstructures built of carbon steel and of mixed nickel steel and carbon steel. With these fifty diagrams at hand and all the necessary conditions given, it is only a minute's work to determine for any case whether it would be more economic or not to adopt nickel steel—also what the saving, if any, in expense would be.

Figs. 3, 4, 5, and 6, chosen at random, are specimens of the said fifty economic diagrams, the first pair being for double-track, through, riveted, Pratt-truss spans, with the price of carbon-steel erected at 4.0 and 4.5 cents per pound, respectively; and the second pair for double-track, through, pin-connected, cantilever bridges of the most usual type, with the price of carbon-steel erected at 4.5 and 5.0 cents per pound, respectively. These prices for carbon-steel erected are about the average ones that govern today in various portions of the United States.

Fig. 7 is an important and interesting diagram. It shows the probable weights of metal per lineal foot of superstructure for very-long-span, double-track-railway, cantilever bridges built of carbon steel and of nickel steel (or of mixed nickel and carbon steels). It indicates also the extreme practicable limit of length of main opening for such bridges for each kind of steel. This limit is a matter of judgment, being determined by the greatest weight of metal per lineal foot of span which it would be advisable to use for the structure under consideration. From the diagram it will be seen that if 1,800 feet be assumed as the present practicable limit of span-length for carbon-steel bridges, the corresponding limit for nickel-steel bridges will be about 2,300 feet; or, if it be assumed at 2,000 feet, the corresponding limit for nickel-steel construction will be 2,600 feet. It is safe, therefore, to conclude that the adoption of nickel steel for bridges would lengthen the practicable span-length for cantilevers fully 500 feet.


COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Lineal Foot of Span.
Span, in Feet
100 110 120 130 140 150 160 170 180 190 200

Figs. 3 and 4.


COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, TYPE A, CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH PIN-CONNECTED, TYPE A
CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND
Cost of Metal Erected, in Dollars per Lineal Foot of Bridge.
Length of Main Opening, in Feet,
300 450 600 750 900 1 050 1 200 1 350 1 500 1 650 1 800

Figs. 5 and 6.

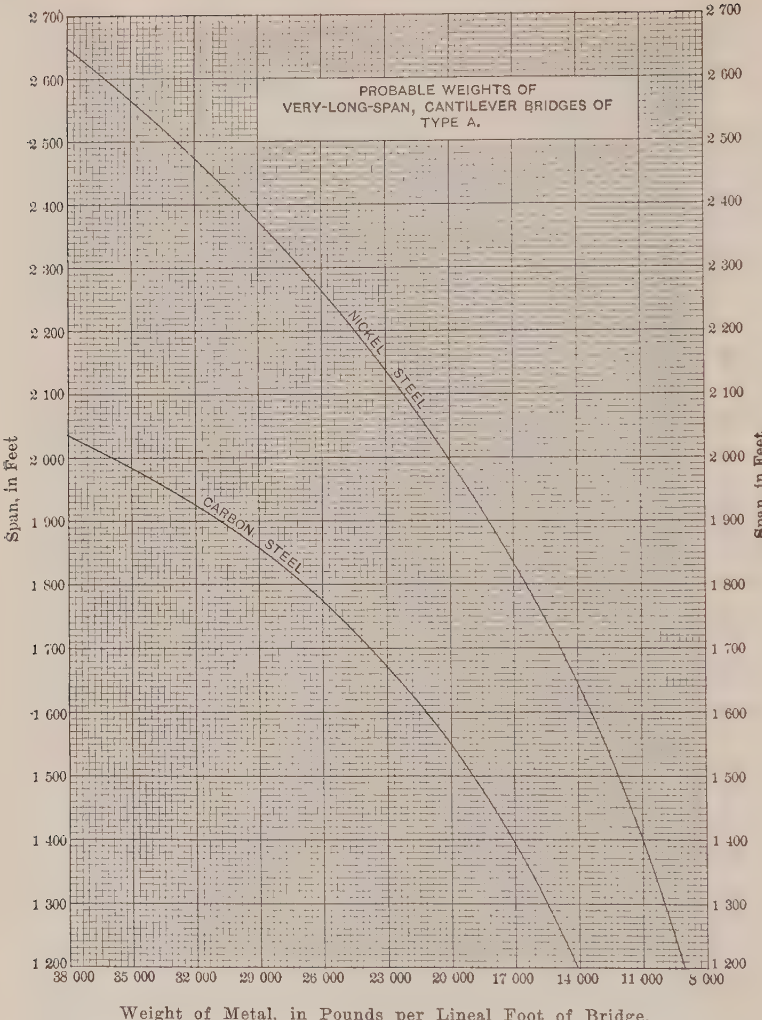


Fig. 7.

In concluding his paper on "Nickel Steel for Bridges" the author wrote as follows:

"Summarizing the results of this entire investigation, it is evident that nickel steel is in every way fitted for bridge construction, in that it is strong, tough, workable, and reliable; moreover, its adoption would effect a decided economy. This economy

would increase in the future as the cost of nickel decreases and as the shops become more accustomed to the fabrication of the new alloy".

The preceding was written in 1907, and the predictions made have been only partially realized; for while, as before indicated, several large bridges have been built of nickel steel, the manufacturers have not been willing to quote reasonable prices for the alloy. If there were anyone available to unload rejections upon, as there is in the case of carbon bridge-steel, the steel makers would quite willingly quote more reasonable figures for nickel steel; but the constant dread of being left with a large lot of unsalable alloy-steel on their hands militates against their so doing. It is only by having engineers create a large demand for the alloy, thus initiating competition in its production, that a reasonable pound price for it can be established. In confirmation of this statement is the fact that when Mr. Hodge called for bids on nickel steel for his great St. Louis Free Bridge, he received and accepted a tender of an excess pound price of 1.65 cents, which is not far from the 1.5 cent limit set by the author in his memoir.

It is claimed by several recognized authorities that it is practicable to produce good and perfectly satisfactory nickel steel by putting ferro-nickel into the charge instead of the metallic nickel, thus avoiding all the expense of refining. This would reduce by two thirds the cost of the nickel content in the alloy; and as that content is the main cause of the high price of nickel steel, it is evident that the employment of ferro-nickel in the smelting would make the cost of the product so reasonable that in a few years it would supplant carbon steel entirely, even for bridges of the shortest spans. Nickel producers used to claim that it is absolutely necessary to employ pure nickel in the smelting, for the reason that ferro-nickel usually contains quite a percentage of copper—a substance totally destructive to steel; but, on the other hand, those who are in shape to put ferro-nickel on the market (and some others also) maintain that the copper and all the other injurious substances contained in the ferro-nickel can readily and cheaply be worked out during the processes of smelting and rolling. Moreover, copper is no longer the bugbear to steel makers that it was a few years ago; for it is now practicable to manufac-

ture good, workable steel containing three (3) per cent of that element. Decidedly, it is of the utmost importance to both the engineering profession and the business world to determine without delay and beyond the peradventure of a doubt whether it is feasible to use, on a commercial scale, ferro-nickel in the manufacture of nickel steel.

There is a fact concerning nickel steel known to the profession, but which, as far as the author can learn, had not until a very short time ago been stated in print, viz., that the Pennsylvania Steel Company has obtained control of an iron mine containing a small percentage of nickel; and is, consequently, able to place upon the market a low grade nickel steel at a reasonable excess cost above that of carbon steel. This steel has been denominated by its makers ''Mayarí Steel''. It is a natural alloy of nickel-chromium steel, containing from 1.00% to 1.50% of nickel and from 0.20% to 0.75% of chromium, with sulphur below 0.04%, phosphorus below 0.03%, and manganese as desired. The carbon range is from 0.03% to 1.50%, depending upon the application of the steel. The ore comes from a deposit of some 25,000 acres at Mayarí in the Province of Oriente on the Island of Cuba. It is estimated that there are 500,000,000 tons of this ore in sight. Mayarí steel is made only by the Pennsylvania Steel Company and the Maryland Steel Company. By a slight modification of the open hearth process it is produced without the necessity of adding alloying elements in the furnace or ladle. Like other nickel steels it offers greater resistance to corrosion than do the ordinary carbon steels.

Desiring to obtain for the preparation of this memoir some authentic information concerning the new alloy, the author wrote to the Pennsylvania Steel Company asking certain questions about it, and received in reply a letter from J. V. W. Reynders, Esq., C. E., the Vice-President of the Company, from which the following is an extract:

> ''Our principal experience on Mayarí steel in bridgework has been in connection with the manufacture and fabrication of the large bridge which is to span the Mississippi River at Memphis. So much steel has now been made for this contract that we have accurate information on the properties which we can develop.

"On this bridge alternate quotations on carbon and alloy steel designs were submitted, the specifications for the alloy steel, outside of rivet and eye-bar steel, being as follows:

Tensile strength	85,000 to	100,000 lbs. per sq. in.
Elastic limit, not less than		50,000 " " " "
Elongation in 8″, not less than		$\frac{1{,}600{,}000}{\text{T. S.}}$
Reduction of area, not less than		30.0%

"The manganese in the steel was limited to .80%, silicon to .15%, carbon to .40%, and a minimum of 1.20% nickel was required, but the individual bidder was allowed to select his own analysis except as it might be limited by these general figures. No limits were given for chromium or vanadium.

"We quoted on the basis of using a steel made from the Mayarí ore which we import from our mines on the north coast of Cuba, and on this basis the contract was awarded us.

"As you doubtless know, this Mayarí ore lies just under a thin top soil in a comparatively thin bed of great area. The ore contains naturally a large amount of moisture, a part of which is in the combined form. It has been our practice before shipping this ore to the United States to run it through rotary nodulizing kilns, which agglomerate the fine ore and drive out the moisture. The nodulized product carries about 57% of iron.

"By a selection of the ore, steel can be produced with a uniform nickel content, which may be varied at will between quite wide limits. It has been found, however, that a content of approximately 1.40% is sufficiently high for bridge steel for most purposes. The steel is normally produced with only the usual additions in the open hearth furnace, although occasionally a small amount of chromium is added.

"The following are typical tests of large size angles, varying from 8″ x 6″ x 1″ up to 8″ x 8″ x 1⅛″:

T. S. Lbs.	E. L. Lbs.	Elong. in 8 In. %	Reduc. of Area %	C. %	Mn. %	Ni. %	Cr. %
95,580	64,700	16.8	45.5	.36	.62	1.27	.36
91,400	60,130	19.5	44.4	.32	.68	1.45	.44
95,440	62,300	18.0	46.7	.34	.75	1.51	.38
94,400	54,300	17.0	50.4	.35	.71	1.48	.43
93,240	55,300	21.5	51.3	.34	.75	1.37	.43
96,740	60,540	16.3	43.3	.32	.75	1.49	.40
98,420	61,060	17.0	48.9	.37	.78	1.31	.42
91,700	56,960	19.0	56.3	.31	.68	1.45	.44
94,180	58,130	20.5	51.5	.30	.77	1.48	.47

The following tests are on plates, both universal and sheared. In many cases you will note that the thickness is extreme.

Thickness Ins.	T. S. Lbs.	E. L. Lbs.	Elong. in 8 Ins. %	Reduc. of Area %	C. %	Mn. %	Ni. %	Cr. %
1½	99,600	64,660	17.5	41.9	.29	.77	1.36	.31
1¼	94,300	62,270	18.0	33.6	.28	.75	1.41	.37
1½	98,820	63,080	17.0	34.7	.28	.72	1.49	.49
⅞	90,040	55,270	21.0	45.5	.28	.67	1.57	.31
1⅛	91,550	62,210	18.5	46.8	.30	.66	1.41	.40
¾	94,060	56,030	18.5	47.1	.27	.61	1.42	.33
1⅜	90,300	57,540	19.0	53.6	.29	.71	1.53	.39

"The phosphorus in all these heats will average less than .02% and the sulphur averages about .03%, the specified limit for each being .04% with an allowance of 25% for check analysis of the finished material.

"In the specifications changes in the elongation and reduction of area are allowed for steel running over 1" thick.

"Below are given the specifications for full size eye-bars and the results of a test on a sample 14" x 1-23/32" bar:

	Required.	Obtained.
Tensile strength.........	80,000 lb. min.	88,200 lb.
Elastic limit..................	47,000 " "	51,700 "
Elong. in 20'...............	10% "	12.7%
Reduc. of area...........	....	42.0%

"It has been our experience that this alloy steel works quite as well in the shops as any other steel with which we are familiar, making due allowance, of course, for the increased toughness. We find that it is easier to work than 3¼% nickel steel.

"It is difficult to give an exact extra which we would charge for rolled sections or plates of Mayarí steel over the market price of similar sections of carbon steel. We do not expect, however, that it will be necessary at any time to charge an extra of more than one cent per pound. The excess price for manufactured bridges depends on so many circumstances that it is almost impossible to give any figure. It will vary greatly, of course, as the relative proportions of carbon and Mayarí steels in the finished bridge are varied.

"With regard to quantity, we expect shortly to be in a position to produce from 18,000 to 20,000 tons per month of Mayarí steel shapes, if necessary; but, even for the present, it is safe to say that we can meet any reasonable demand."

Judging from Mr. Reynders' approximate quotation for the rolled metal and from previous experience with nickel steel, the author concludes that the finished metalwork is likely to cost as much as one and a half cents per pound in excess of the market price of corresponding carbon steelwork. This is just what he first estimated would be the limiting excess pound price of nickel steel having an elastic limit of 60,000 lbs., when the excess price of the rolled material was one cent per pound.

The large-scale curves from which were prepared the cost diagrams of the paper on "The Possibilities in Bridge Construction by the Use of High Alloy Steels", hereinafter referred to at length, afford a means of determining the economics of Mayarí steel for bridges as compared with nickel steels of 55,000 lbs. and 60,000 lbs. elastic limit. From them are found the following:

500-ft. Simple Truss Spans.

The Mayarí steel bridges at 1.5 cts. per lb. excess over carbon steel are equal in cost to nickel steel bridges for E. L.=55,000 lbs. at an excess of 1.9 cts. per lb., and to nickel steel bridges for E. L.=60,000 lbs. at an excess of 2.25 cts. per lb. With Mayarí steel at 1.0 cts. per lb. excess over carbon steel, the corresponding figures are, respectively, 1.35 cts. and 1.7 cts.

For equal costs of bridges, as compared with carbon steel, Mayarí steel could stand an excess pound price of 2.1 cts. for the manufactured superstructure.

1000-ft. Simple Truss Spans.

With Mayarí steel at 1.5 cts. per lb. excess, the excess for nickel steel of E. L.=55,000 lbs. is 2.25 cts. per lb. and that for nickel steel of E. L.=60,000 lbs. is 3.0 cts. per lb. With Mayarí steel at 1.0 cts. per lb. excess over carbon steel, the corresponding figures are, respectively, 1.7 cts. and 2.4 cts. per lb.

For equal costs of bridges, as compared with carbon steel, Mayarí steel could stand an excess pound price of 3.75 cts. for the manufactured superstructure.

Cantilever Bridges with Openings from 1000 ft. to 2000 ft.

Mayarí steel bridges at 1.5 cts. per lb. excess over carbon steel are equal in cost to nickel steel bridges for E. L. = 55,000 lbs. at an excess of 2.3 cts. per lb. and to nickel steel bridges for E. L.=60,000 lbs. at an excess of 3.1 cts. per lb. With Mayarí

steel at an excess of 1 ct. per lb., the corresponding excesses for the other steels would be, respectively, 1.7 cts. and 2.5 cts. per lb.

For equal costs of bridges, as compared with carbon steel, Mayarí steel could stand an excess pound price of 1.85 cts.

From the preceding it is evident that Mayarí steel has carbon steel beaten for bridgework under all conditions, but that if it costs when manufactured 1.5 cts. per pound more than that metal, it will probably not be as economic as either of the grades of nickel bridge steel which can be produced commercially today. If, however, the manufacturers of Mayarí steel and of structures made therefrom can bring the price of their finished metalwork down to an excess of one cent per pound as compared with carbon steel, their product will have somewhat more than a fighting chance in the competition. Nevertheless it will always have one serious obstacle to contend against, viz., the irregularity of the composition and characteristics of the finished product. This is shown very clearly in Mr. Reynders' letter; for in his shape-steel tests the elastic limit varies from 54,300 to 64,700 pounds per square inch, the ultimate strength from 91,400 to 98,420 pounds per square inch, the nickel content from 1.27 to 1.51 per cent, and the chromium content from 0.36 to 0.46 per cent. In the plate tests the corresponding variations were, respectively, from 55,270 to 64,660 pounds per square inch, from 90,040 to 99,600 pounds per square inch, from 1.36 to 1.57 per cent, and from 0.31 to 0.49 per cent. Considering that the raw material received very little preparation for smelting, the preceding showing is by no means bad, especially since the records given indicate that no special difficulty has been experienced in complying with the specifications. On the other hand, though, the serious disadvantage under which the alloy labors is strikingly made evident by averaging the elastic limits given in the specimen tests; because the mean of all the figures is 59,655 lbs., while the requirement was only 50,000 lbs. It is possible that experience in the production of the alloy will result in greater regularity and less cost. If such proves to be the case, Mayarí steel is likely to supplant entirely the other alloy bridge steels at present obtainable; but it is far from being the ideal alloy for long-span bridge construction. Even if the inherent irregularity be made truly non-injurious to the metal by al-

ways keeping its characteristics well above the specified requirements, there will (for many years, at least) exist in the minds of purchasers the latent doubt of the steel's reliability and the dread that, without warning, the elastic limit and the ultimate strength may drop dangerously below the minima called for in the specifications. For a long time to come, and perhaps always, it will probably be necessary to test Mayarí steel much more thoroughly than carbon steel in order to prevent the utilization of any inferior melt or rolling in the manufacture of bridgework.

In the development of Mayarí steel for bridgework, credit is due to Mr. Ralph Modjeski, M. Am. Soc. C. E., the Consulting Engineer on the new Memphis Bridge, the first large structure in which that alloy is to be used.

During a stay of some six weeks in France in 1909, the author learned that certain metal manufacturers in that country were making, in melts of five tons or less, by the electro-metallurgical process, a purified steel for which they claimed rather astonishing results in respect to high elastic limit, great ultimate strength, and general suitability for the manufacture of bridges; although, as far as the author could ascertain, no such structures up to that time had been built of the new product. It was not convenient for him then to obtain and test specimens of the steel, as he greatly desired to do; hence he had to content himself with second-hand information obtained by both interviews and correspondence. The results of these convinced him that the claims made might, at least partially, be justified by performance; thereupon, having some spare time, he prepared an economic study of the possibilities for utilizing such purified steel in bridges. In his calculations he employed French units, prices, and other conditions, publishing the results in French in a memoir for *Le Génie Civil* under the title, *"Etude Economique de l'Emploi de L'Acier au Carbone à Grande Résistance, pour la Construction des Ponts."*

The French metallurgists, steel manufacturers, and bridge engineers, to whom the author applied for information were all most kind and courteous in furnishing it, enabling him to collect quickly all the general data needed. Just here the author claims the privilege of expressing publicly his high appreciation of the exceeding kindliness and courtesy which French engineers and

French scientists make a practice of showing towards their professional brethren from the United States. Nothing seems to give them too much trouble to do as accommodation; and they are ever ready to devote hours of their valuable time to discussing the similarities and differences between French and American conditions, practice, and customs in all matters of a technical nature.

The excess cost of the French purified steel, as compared with the ordinary carbon bridge-steel of that country, appeared to be about nine tenths of a cent per pound for the manufactured superstructure. The investigation showed the economics for its employment in bridge building for the mean and the extreme conditions of the French metal market, and for a number of assumed elastic limits, varying from 30 to 45 kg per sq. mm, the value for the usual carbon bridge-steel in France being 24 and that for the author's specified nickel steel 42.5 kg per sq. mm. The outcome of the investigation was that there was found no advantage whatsoever for the 30 kg elastic limit; none for short spans but a small one for long spans with a 35 kg elastic limit; a decided saving for all cases with a 40 kg limit; and a wonderful economy for the 45 kg limit, the highest claimed by any of the French manufacturers.

Figs. 8, 9, 10 and 11 are taken from the issue of *Le Génie Civil* dated Aug. 7, 1909. They show for carbon steel, the author's specified nickel steel, and the purified steels having assumed elastic limits of 30, 35, 40, and 45 kg per sq. mm, respectively, the weights of metal in kilogrammes per lineal meter for simple-span bridges, ditto for cantilevers, the costs in francs per lineal metre of span of the steel erected in simple-span bridges, and the same in cantilever bridges of the most usual type. In Figs. 10 and 11 the assumed condition of the carbon-steel market was that which existed in France at the time the investigation was made. Most fortunately, it was also the exact mean of the two extreme conditions.

It was the author's hope that the publication of his paper would give an impetus in France (and perhaps elsewhere also) to the manufacture of bridges of purified steel; but the hope has proved to be a vain one; for, up to the present, he has not heard of any such development. It is probable that the metallurgists

and the bridge manufacturers of France are no more eager to adopt drastic innovations in their practice than are their brethren in the United States.

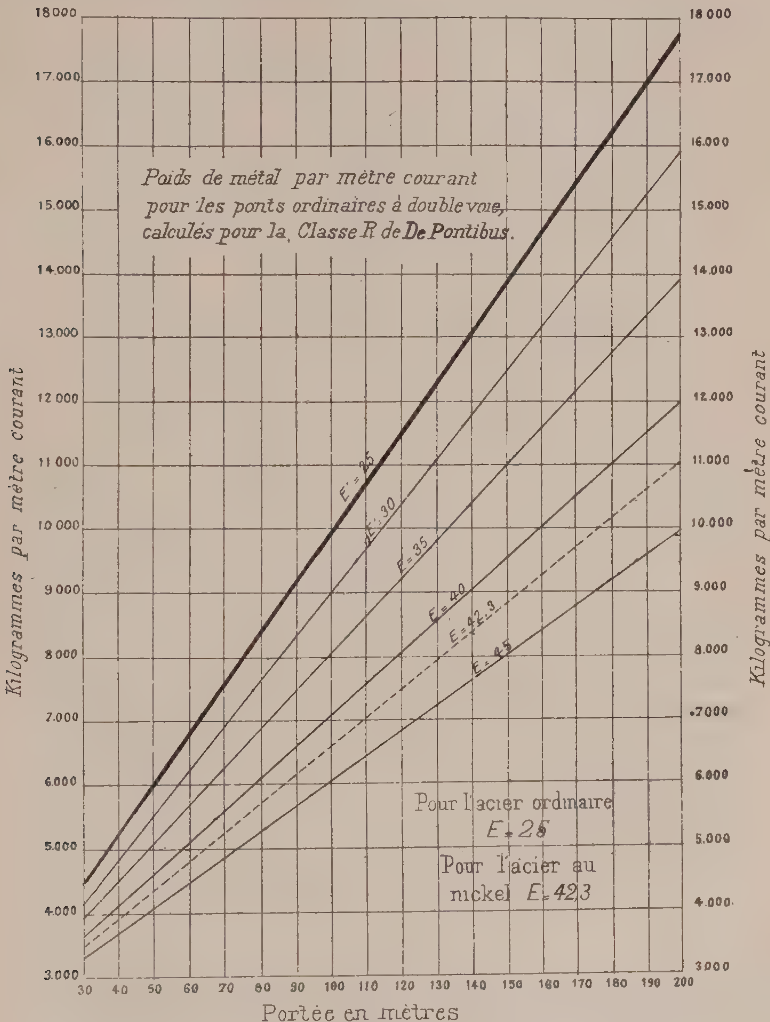


Fig. 8.

If nothing ever comes of that investigation, and if purified steel is never used directly for bridge building, it is within the

realm of possibility that the ideal future alloy of steel for bridges will be made by first purifying carbon steel, either by the electro-metallurgical process or by some other method, before the alloying element is added to the molten mass, in which case all the

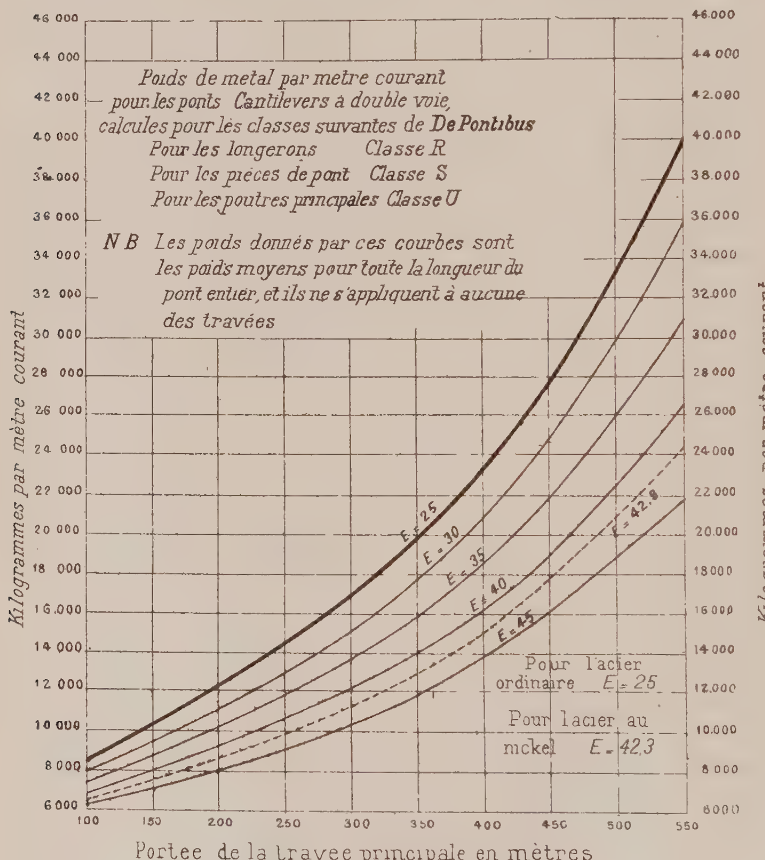


Fig. 9.

trouble that the author went to in preparing the paper just described would not be wholly wasted.

Desiring to obtain for the preparation of this paper some authentic information concerning the status of the manufacture of purified steel, the author consulted the United States Steel Cor-

poration on the subject; and in reply to his letter received a communication from W. R. Walker, Esq., the Assistant to the President of that Company, dated April 28, 1914, from which the following extract is quoted:

> "Although the electric steel process is a comparatively new one, being only ten years old, nevertheless, there are in operation

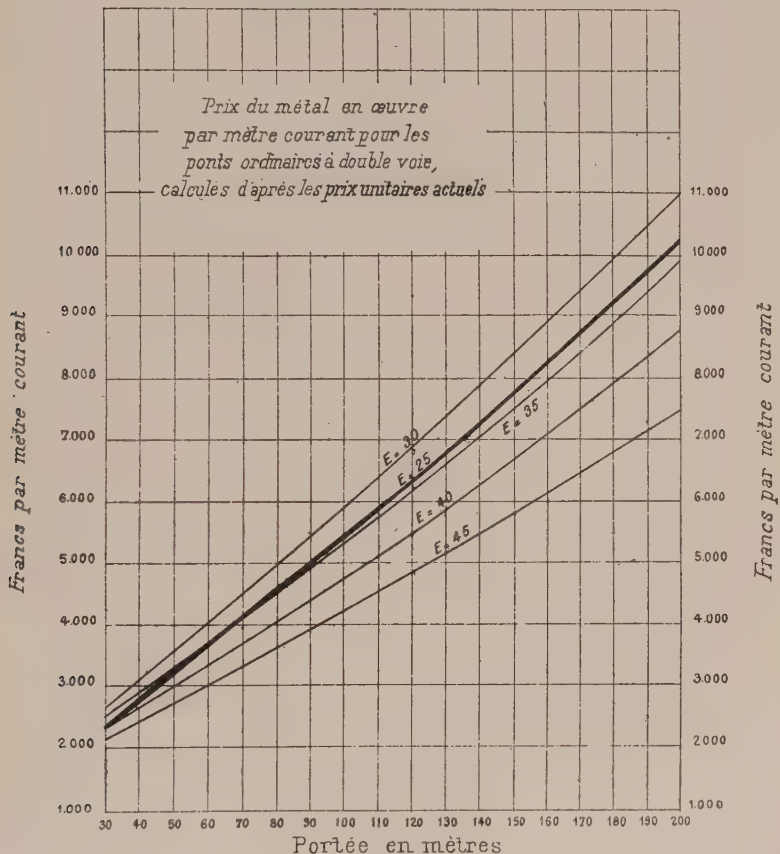


Fig. 10.

> about 130 electric furnaces in this country and Europe which are making high grade steel commercially. This steel has proven to be of such excellent quality that it is rapidly displacing Crucible Steel, which is generally considered the standard of excellence. It is also being used in seamless tubes, wires, sheets, ship angles, rails, thin armor plate, and especially tools.

"In 1909 the Steel Corporation began the operation of a 13-ton electric furnace at the South Chicago plant of the Illinois Steel Company, and in 1910 began the operation of a similar furnace at the Worcester Works of the American Steel and Wire Company. These installations were, at the time, experimental to the extent that it was not known if we could make the heavier

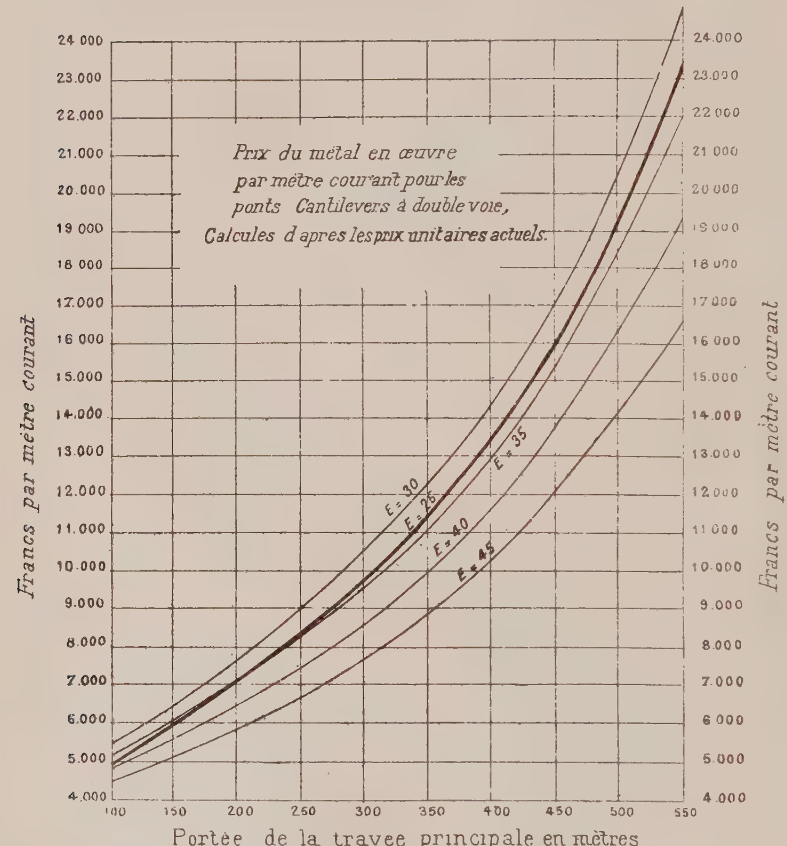


Fig. 11.

products, such as rails, commercially. We have made about 10,000 tons of rails which are now in track. A number of years are required to test out the value of rails; but, up to this time, none of our electric steel rails have broken in service—even those located in the far Northwest during the very severe winter of 1912.

> "Although the electric furnace at Worcester has not been in operation for some time, due to commercial reasons, the furnace at South Chicago has operated almost continuously ever since its installation, and is now employed in making alloy steels, such as nickel-chrome, chrome-vanadium, high silicon steels, etc.
>
> "From these facts I think you can safely conclude that electric steel is no longer an experiment and that its quality is equal to the best made by any other process".

Late in 1913, not being satisfied with the progress then being made in the use of special steels of high elastic limit for long-span bridges, the author prepared for the American Society of Civil Engineers a paper entitled "The Possibilities in Bridge Construction by the Use of High-Alloy Steels". It was printed in the Society's Proceedings early in 1914, and has since been discussed by some sixteen engineers of the United States, Europe, and other parts of the world, and is now about to be published in the Society's Transactions.

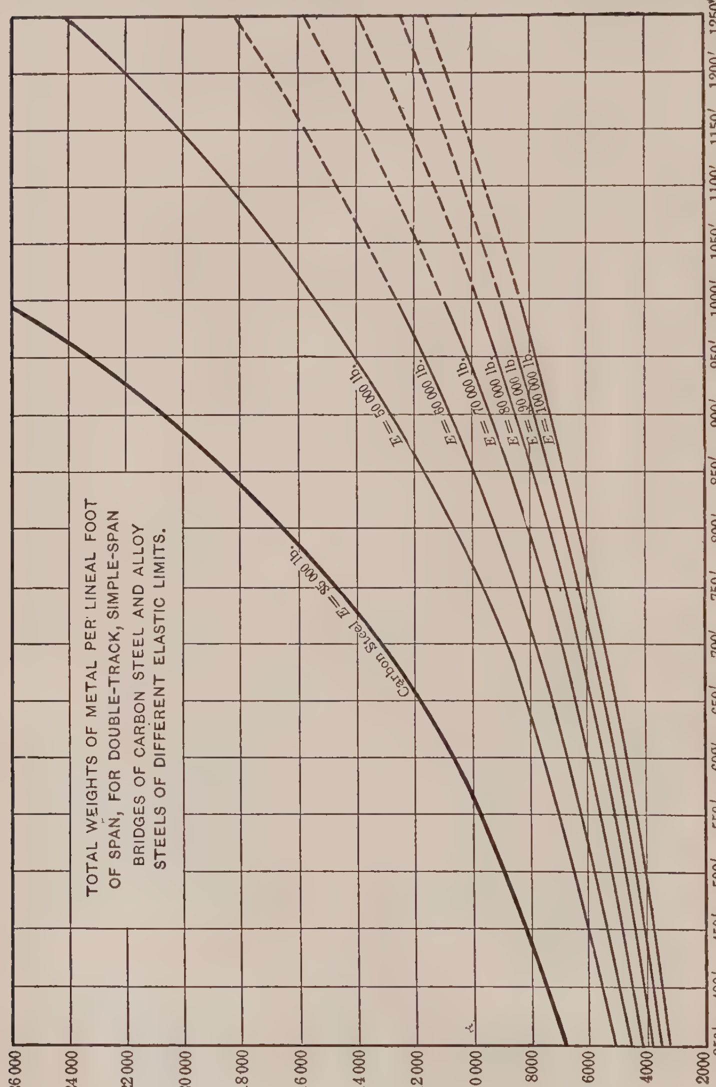
As might be anticipated from the title of the memoir, its object is to determine, for the usual types of bridges and for all practicable span-lengths, the weights of metal per lineal foot of structure that would be required when using alloy steels of varying elastic limits, and the economics involved by their employment. Incidentally, there would be found the extreme practicable limits of span-length for cantilever bridges constructed for the greater part of such materials.

The elastic limits assumed varied by 10,000 lbs., starting with 50,000 lbs. and ending with 100,000 lbs. Fig. 12 gives the weights of metal per lineal foot of span for double-track, simple-span bridges, and Fig. 13 records those for double-track, cantilever bridges. In the text of the memoir are given directions for finding the corresponding weights for similar bridges having more than two tracks and for those carrying other live loads than the ones assumed in the investigation.

From Fig. 12 it is evident that in simple-span structures there is an immense saving in weight of metal by using alloy steel instead of carbon steel, also that the rate of saving diminishes gradually as the elastic limit of the metal increases.

In Fig. 13, the saving of material by employing alloy steels, while not quite so striking as in the case of Fig. 12, is still most apparent.

If, as can be seen by Fig. 13 to be logical, it be assumed that a limit of 36,000 lbs. of metal per lineal foot of span is as high



Weights of Metal, per Lineal Foot of Span, in Pounds.

Span Lengths

Fig. 12.

as it is either economical or practicable to go in the building of double-track, railway, cantilever bridges, the corresponding lim-

iting lengths of main openings will be approximately as follows:

For carbon steel, E = 35,000 lbs.	2030	Feet
" E = 50,000 lb. steel	2340	"
" E = 60,000 " "	2590	"
" E = 70,000 " "	2780	"
" E = 80,000 " "	2910	"
" E = 90,000 " "	3030	"
" E = 100,000 " "	3140	"

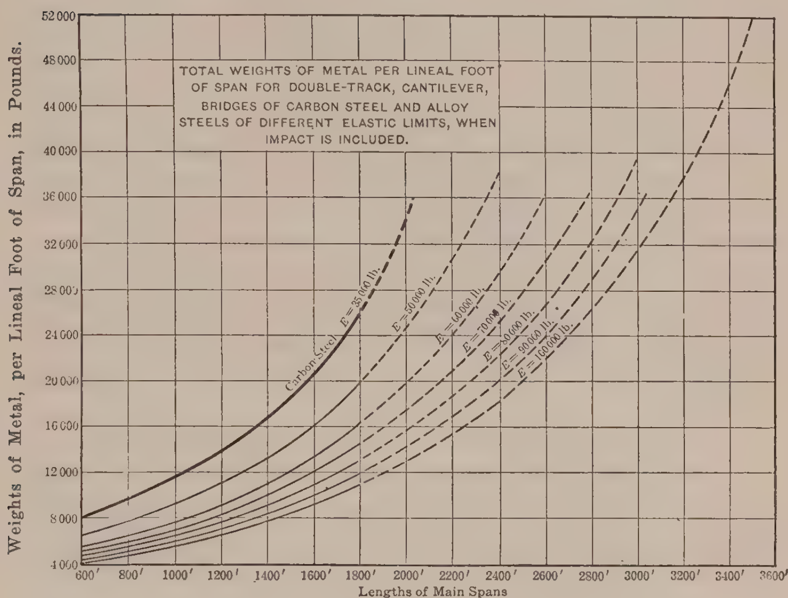


Fig. 13.

From the appearance of the curves at their superior ends one may draw the conclusion that, in the case of the very-high-alloy steels, the limit of weight of metal per lineal foot of span can legitimately be raised beyond the previously assumed 36,000 lbs. The more nearly these curves approach the vertical the more uneconomical would it be to extend the limit beyond the said 36,000 lbs. per lineal foot.

In studying the economics of the various alloy steels, the present ruling pound prices for carbon steel bridges erected were assumed to be 4.5 cents for simple spans and 5.0 cents for cantilevers.

Fig. 14 is a specimen of the economic diagrams for simple-truss bridges. It shows that, even with an excess pound price of 4.5 cents for the fabricated alloy metal, there is a saving over carbon steel when the elastic limit of the alloy is 80,000 lbs. per square inch. Fig. 15, which is for cantilevers and for an elastic limit of 80,000 lbs. per square inch, shows that the same conclusion holds as that just drawn for simple-truss spans.

In concluding the memoir the author says it is evident that his results clearly prove that a systematic series of experiments made in search of a suitable and satisfactory alloy steel for building long-span bridges would be well worth while. He indicates that he is of the opinion that the first step to take in such an investigation would be to experiment on "purified" steel so as to bring it to its maximum of effectiveness, then to try adding nickel in various quantities, and afterwards nickel with other but cheaper substances. He recognizes that augmenting the carbon in the purified steel, while increasing both its ultimate strength and its elastic limit, would tend to harden the metal, but anticipates that the addition of nickel (and possibly other elements) would tend to reduce the brittleness and render it workable.

The concluding paragraph of his paper reads as follows:

"The problem of finding a high, cheap alloy of steel, suitable in every particular for bridges, is now before the metallurgists and the builders of large metallic structures; and the values of all the results probably attainable are clearly indicated in this paper; hence the onus is on the Engineering Profession to see that the necessary experiments are arranged for and thoroughly carried out, in order that the world may have at its command a new metal that will permit of the spanning of waterways which are so wide and so deep, or are so restricted by navigation requirements, as at present to defy the art of the bridge engineer".

In the sixteen discussions of the paper there were advanced two pertinent suggestions concerning how to find the desired alloy. One was to use about three (3) per cent of aluminum as the principal alloy element, the present price thereof being only 20 cents per pound; and it was anticipated that such a combination might produce a satisfactory steel having an elastic limit of 100,000 lbs. per sq. in. It was evident from the way in which

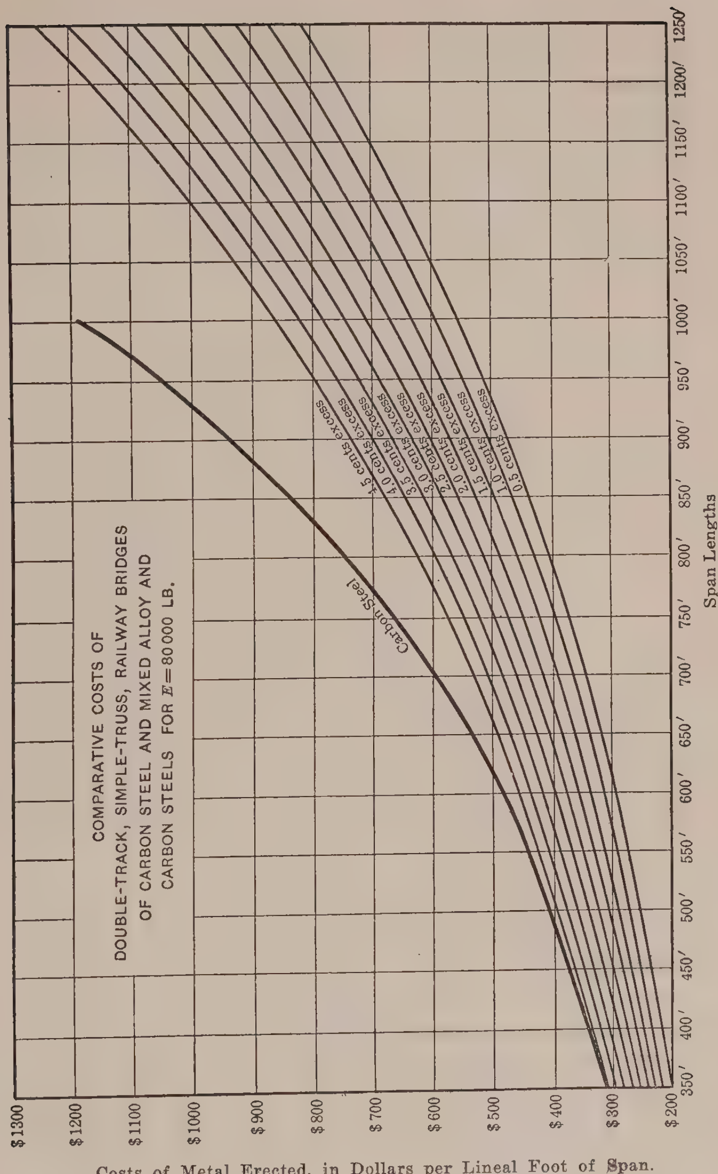

COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR $E=80\,000$ LB.
Carbon Steel
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350′ 400′ 450′ 500′ 550′ 600′ 650′ 700′ 750′ 800′ 850′ 900′ 950′ 1000′ 1050′ 1100′ 1150′ 1200′ 1250′
Span Lengths
$1300 $1200 $1100 $1000 $900 $800 $700 $600 $500 $400 $300 $200
Costs of Metal Erected, in Dollars per Lineal Foot of Span.

Fig. 14.

the discussion was worded that no experimenting worth mentioning had been done on that alloy for the purpose of bridge building; hence the suggestion must be treated as a wholly tentative one, although decidedly alluring.

The other suggestion, made by Geo. L. Norris, Esq., Metallurgical Engineer of the American Vanadium Company of Pittsburgh, was that either vanadium-carbon steel or vanadium-chromium steel be used as a high-alloy for bridgework. This suggestion was much more directly to the point than the other;

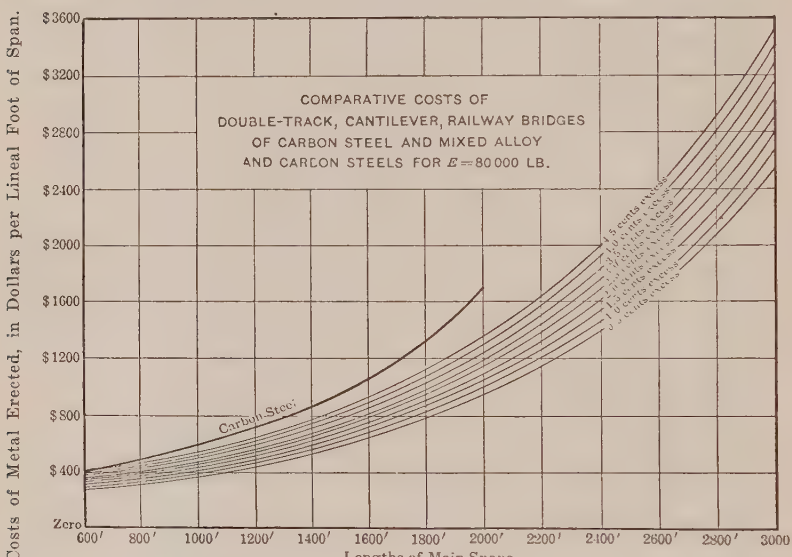


Fig. 15.

because both of the vanadium alloy steels mentioned have been manufactured for several years, although not for the purpose of bridge building. Mr. Norris was able to give the chemical and the physical qualities of the alloys recommended, but, unfortunately, only by stating very wide limits therefor.

In answer to a long list of questions concerning the use of vanadium in steel, propounded by the author in a letter to the American Vanadium Company, and incorporated as a part of the résumé of discussions, the following important information was obtained from Mr. Norris:

A. Vanadium steel is eminently fitted for the manufacture

of eye-bars. The elastic limit for full-size, chrome-nickel-vanadium bars varied in the tests from 63,280 lbs. to 80,480 lbs., and the ultimate strength from 93,000 lbs. to 99,800 lbs., the results depending upon the drawback or annealing temperature after quenching. The excess cost per pound of these finished bars after treatment, as compared with ordinary eye-bars of carbon steel, Mr. Norris indicated would not exceed 3.5 cents.

He anticipated that simple carbon-vanadium steel eye-bars would have an elastic limit of from 60,000 to 75,000 lbs., and an ultimate strength of from 85,000 to 100,000 lbs.; but it is evident that no experiments have been made on finished bars of that kind. The excess cost of such eye-bars after annealing, as compared with ordinary eye-bars of carbon steel, would not be more than 1.5 cents per pound.

B. It was not quite so evident from the discussion that vanadium steel is as suitable for built members of bridges as it is for eye-bars, but it seems probable that it would be found satisfactory.

Mr. Norris states that his chrome-vanadium steel will be workable under shop manipulations.

C. It appears from Mr. Norris' remarks that heat-treated vanadium steel can be manufactured into built members of bridges without losing the great effect of the treatment, but it would probably be better to drill the rivet holes solid than to sub-punch and ream them. Mr. Norris did not answer the question whether vanadium steel can be bent cold without injury; hence one may surmise that it cannot. However, this would not militate greatly against its use in bridgework, because in long-span structures (the only kind now under consideration) there should be very little, if any, metal to be bent.

D. In respect to rivets, Mr. Norris advises, for chrome-vanadium steel, E. L. = 50,000 to 65,000 lbs., and Ult. = 70,000 to 90,000 lbs.; and for simple carbon-vanadium steel E. L. = 40,000 to 55,000 lbs., and Ult. = 65,000 to 85,000 lbs. The author is of the opinion that some serious difficulty might be encountered in cutting out defective rivets as high in strength as those first mentioned.

Concerning rivets for bridges built of high-alloy steels, it is a foregone conclusion that the ratio of the strength of the alloy-

steel rivets to that of carbon-steel rivets cannot be as great as the corresponding ratio of strength of plate-and-shape alloy steel to that of plate-and-shape carbon steel. On this account, in high-alloy-steel bridges it will be necessary to use proportionately either more rivets or greater rivet diameters—or both.

E. The amount of chromium recommended by Mr. Norris varies from 0.6% to 0.9% in combination with manganese varying from 0.4% to 0.6%, or even to 0.8%.

F. Although for a number of years it was thought by the engineering profession in general that vanadium in steel acts merely as a scavenger, none of it remaining in the finished product but all of it passing off with the slag, Mr. Norris asserts that about 80% of the vanadium which is added to the charge remains in the metal.

Mr. Norris is positive that the vanadium is very evenly distributed through the ingot, and that not only is there no danger whatsoever of its segregation, but also that its presence in the molten metal tends to prevent the segregation of other substances—notably carbon. This is most reassuring and is a strong point in favor of the employment of vanadium steel for bridge building.

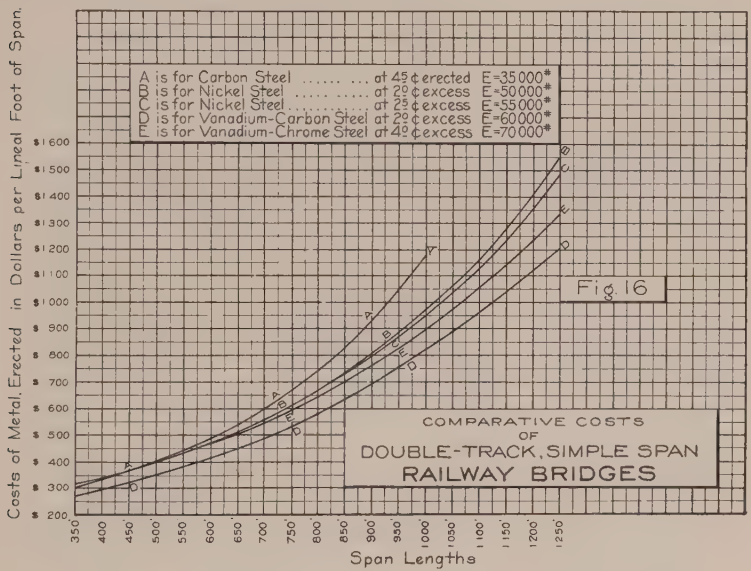
G. From what Mr. Norris states, one might anticipate that the gain by adding nickel to vanadium-carbon steel or to vanadium-chromium steel would be either very small or non-existent.

H. Mr. Norris is quite convinced that the addition of titanium to vanadium steel would be useless. Perhaps the explanation for this is that the 20% of the vanadium charge which passes off into the slag acts as a scavenger, thus obviating the necessity for any further purification—which office is the sole function of the titanium.

The author's assumption from Mr. Norris' data of E. L. = 60,000 lbs. for his vanadium-carbon steel and E. L. = 70,000 lbs. for his vanadium-chromium steel, with excess pound prices for the manufactured metals of 2.0 cents and 4.0 cents, respectively, would establish specifications and prices that the steel makers and bridge manufacturers should have no special trouble in living up to. It is possible that these assumptions do not do sufficient justice to the vanadium steels, but the figures had to be made safe for an investigation of the economics of the two types

of vanadium steel compared with carbon steel and with the two classes of nickel steel which are procurable today in the United States for bridgework, viz., that for E = 50,000 lbs. at an excess pound price of 2.0 cents and that for E = 55,000 lbs. at an excess pound price of 2.5 cents.

Under the preceding conditions the economics of the five steels considered are shown for simple-truss spans in Fig. 16 and for cantilever bridges in Fig. 17.



Referring to Fig. 16, it is seen that for simple-span bridges, with the conditions assumed, vanadium-carbon steel shows, for all span-lengths, a small but material advantage over all the other steels; and that the vanadium chromium steel begins to develop an economy over carbon steel at a span-length of about 500 feet, and over the two nickel steels at a span-length of about 650 feet.

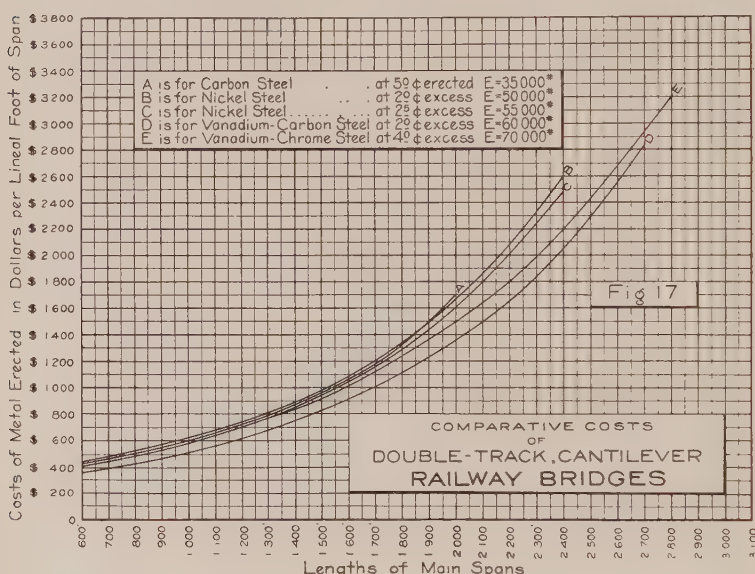
Referring to Fig. 17, it is seen that for cantilever bridges, under the conditions assumed, vanadium-carbon steel shows for all span-lengths, as in the case of simple-truss spans, a small but material advantage over all the other steels; and that the vanadium-chromium steel begins to develop an economy over the

carbon steel and the two nickel steels when the main opening has a length of about 1400 feet.

The author closes the résumé of the discussions of his paper on "The Possibilities in Bridge Construction by the Use of High-Alloy Steels" in these words:

"Summarizing the findings of the discussion and the résumé, there can be drawn the following conclusions:

"First. Titanium as a scavenger of carbon steel promises good and useful results at exceedingly low cost. While it does



not increase greatly the elastic limit or the ultimate strength of the metal, it makes it much more uniform and reliable. On that account it should be employed in a few cases on bridgework; and then, if it be found satisfactory, its adoption should be made obligatory by the railroad companies and the other builders of carbon steel bridges.

"Second. There appears to be a great possibility in the use of aluminum as an alloy for bridge steel; but, as far as the author can determine, very few experiments in aluminum steels have yet been made; hence the said possibility is more or less hypothetical.

"Third. The possibility of obtaining a good, high-alloy steel for bridges by the use of vanadium appears to be almost a certainty; but the highest elastic limit and ultimate strength which can be obtained upon a commercial basis by the use of that element cannot be determined without making some elaborate and exhaustive experiments".

In closing this memoir for the International Engineering Congress, the author feels that he ought to conclude with an apology to such an august assembly for the inherent personality of practically its entire substance—and such an apology is herewith tendered with all due respect. In extenuation of his transgression, however, he would state that, in relation to the subject of "alloy steels in bridgework", he has been so closely connected with the matter ever since its inception that he feels justified in applying to his case the words of the renowned Æneas, "*et quorum pars magna fui*".

ADDENDUM.

SILICON STEEL.

There is being used for the main built-members of the trusses of the C. B. & Q. R. R. bridge, now in process of construction, across the Ohio River at Metropolis, Ill., an alloy that has been termed "silicon steel". Its specific composition and characteristics are as follows:

Phosphorus, max. (basic)	0.04%
Phosphorus, max. (acid)	0.06%
Sulphur, max.	0.05%
Carbon, max.	0.40%
Manganese, max.	1.00%
Silicon, min.	0.25%
Ult. Strength, min.	80,000 lbs.
Ult. Strength, max.	95,000 lbs.
Yield Point, min.	45,000 lbs.
Percentage of elongation in 8 inches = 1,600,000 ÷ Ult. Str.	
Reduction of area, min.	35%
Fracture desired	silky
Cold bend without fracture,	
¾ inch thick and under	d = t
¾ to 1½ inches thick	d = 1.5t
over 1½ inches thick	d = 2.5t

The excess cost of the rolled silicon steel, as compared with the ordinary carbon bridge steel, is one half cent per pound, and the excess cost for shopwork is about 0.15 cent per pound.

The author is curious to know what would be the effect on the above steel if the twenty-five points of silicon were either omitted altogether or reduced to a minimum. His surmise is that very little difference would be noted; for, as stated in Chapter III, ordinary carbon steel containing forty points of carbon and about sixty-five points of manganese has an ultimate strength of 80,000 lbs., which would be increased somewhat by putting in thirty-five additional points of manganese.

The high steel for bridges specified by the author nearly two decades ago in De Pontibus, but never used by him in any of his work, was as follows:

Phosphorus, max. (acid)	0.07%
Sulphur, max.	0.05%
Silicon, max.	0.06%
Manganese, max.	0.80%
Ult. Strength, min.	70,000 lbs.
Ult. Strength, max.	80,000 lbs.
Yield Point, min.	40,000 lbs.
Percent elongation in 8 inches	from 16 to 20%
Reduction of area	from 30 to 38%
Fracture desired	silky
Cold bend without fracture	$d = 2t$.

Such bridge metal is not greatly inferior to the silicon alloy just described; and if the permissible manganese were increased from the specified 80 to the 100 points permitted in silicon steel, the differences in resistance characteristic of the two steels would be somewhat reduced. Again, the specifications for the silicon steel require all rivet holes to be drilled from the solid—an old requirement for high-carbon bridge steel.

From what precedes, the author is led to the conclusion that the "silicon steel" of the Metropolis Bridge is merely a slightly improved form of high-carbon steel, notwithstanding the fact that his friend, C. W. Bryan, Esq., C. E., Chief Engineer of the American Bridge Company, who furnished him with all of the preceding data that are not given in Engineering News of July 29, 1915, is of the opinion that silicon steel suitable for bridge-

work may be obtained later on with a greater elastic limit than 45,000 lbs. per square inch.

From a commercial point of view silicon steel with E = 45,000, and at an excess pound price, compared with carbon steel, of one-half cent for the rolled raw material, is a close competitor of Mayarí steel with E = 50,000, and at a corresponding excess pound price of one cent. To prove the correctness of this statement, let us compare the costs of Mayarí steel and silicon steel, long-span, simple-truss, double-track bridges, erected, when the corresponding carbon-steel bridges in place are worth 4.5 cents per pound. The weights of metal for an assumed span length of 750 feet, taken from Fig. 12, are, respectively, 9650 lbs. and 10,350 lbs. per ft. of span. Neglecting, for convenience, the small variation of cost per pound involved in the erection, and assuming, as in the body of the paper, a total excess of 1.5 cents per lb. delivered at bridge site for the Mayarí steel, we have the following comparison:

Mayarí steel, 9,650 lbs. at 6c.	$579.00
Silicon Steel, 10,350 lbs. at 5.5c.	569.25

Difference in favor of silicon steel........$ 9.75 per lineal foot of span, or about 1.7 per cent.

VANADIUM CARBON STEEL.

Since the time when Mr. Norris furnished the information concerning vanadium steel that is contained in the body of this paper, he has had made two melts of vanadium carbon steel which are of interest to bridge engineers, although the metal was intended for other purposes than bridge building. The results of these tests, which are by no means conclusive, and which, in their incompleteness, may properly be deemed somewhat disappointing, nevertheless hold a promise of the eventual attainment of the desired alloy. But to accomplish this, future tests must be made systematically and in accordance with a programme based on a thorough study of the problem by an expert bridge engineer who has in mind the needs of the advancing art of bridge construction and the possibilities awaiting it, if a high-elastic-limit alloy-steel can be had. If such an investigation were made, it might be practicable to discover a workable

and perfectly satisfactory steel alloy of vanadium (and perhaps other special elements) for the manufacture of bridges. With a report from a recognized and disinterested authority, the engineering profession would be quick to see and avail itself of the advantages offered by this superior bridge metal. The author would be content if there were obtained an alloy, suitable in every particular for bridge construction, having an elastic limit of 80,000 lbs. per square inch for annealed eye-bars of the largest size, and one of 70,000 lbs. per square inch for plate-and-shape steel; but it might be within the realm of possibility to raise the last figures as much as 5000 lbs., especially as solid drilling can now be adopted instead of sub-punching and reaming without adding materially to the cost of the shopwork. The author is of the opinion that such results can be obtained by means of vanadium, but only, as just mentioned, after a thorough and exhaustive investigation shall have been made by an expert bridge engineer. Such a person should be allowed ample funds for all legitimate expenditures involved in the exact determination of the different properties of a number of trial alloys of various compositions. Only in this way can the economics of the problem be settled. A picayunish policy adopted in the making of any important economic investigation is more than likely to defeat its purpose, and it is absolutely certain to involve a greater or less falling short of ultimate possible success in the complete attainment of the desired information.

DISCUSSION

Mr. Reynders. **Mr. J. V. W. Reynders,*** M. Am. Soc. C. E. (by letter), stated that the Mayarí steel which Mr. Waddell referred to is a nickel-chromium steel in which the alloys are present as a result of the use of ore containing them instead of by introducing the alloying metals in the open-hearth furnace or in the ladle. On account of the source of the alloying elements in Mayarí steel, it is obvious that there is less likelihood of a non-uniform dissemination of the alloys in an open-hearth heat than in steel to which lumps of almost pure metallic alloys must be added to secure the desired content.

Nickel-chromium steel, both natural alloy Mayarí and synthetic alloy steel, is probably the most widely used alloy steel at the present time, and although it was first used largely in automobile work, it is rapidly extending into other lines. So many tests have already been

* Vice-president, Pennsylvania Steel Co., Steelton, Pa.

made upon it and it has been used to such an extent in various kinds of the most important work, that there is no question of its safety and reliability. Mr. Waddell's remarks, however, might leave an impression that Mayarí steel in particular is liable to unusual variations in chemical composition or physical characteristics. The figures which he mentions as illustrating this point, as a matter of fact, indicate quite the reverse. For instance, a variation in the ultimate tensile strength from 91,400 to 98,420 lbs., a total difference of 7020 lbs. for steel shapes, and from 90,040 to 99,600 lbs., a total difference of 9560 lbs., for plates running from ¾ in. to 1½ in. in thickness, is about as little as anyone can reasonably expect from any grade of steel, and on this high tensile steel it is much less in proportion than is ordinarily expected and furnished in the standard carbon steels which are used for bridgework. The same remarks will apply to the variations in elastic limit for shapes from 54,300 to 64,700 lbs., a total difference of 10,400 lbs., and on plates from 55,270 to 64,660 lbs., a total range of 9390 lbs. On the chemical analysis it is common practice to allow a range of ½ of one percent between the lower and upper limits for nickel and 0.3 of one percent for chromium. The greatest variation mentioned by Mr. Waddell, however, is 0.30% for nickel and 0.18% for chromium. Mr. Reynders.

It is unquestionably true that when using high-grade steels in the cases where safety depends not so much on the mass of steel as on its high quality, more and severer tests should be made than on the more common grades of material. This is a general condition, however, applying to all high-grade steels and not alone to Mayarí steel, as indicated by Mr. Waddell.

Alloy steels are finding their place in all the more important lines of work, and statistics will show that their production, year by year, is increasing rapidly. In this respect bridgework is no exception to the general rule, and Mr. Waddell's paper, therefore, is a very timely contribution on present practice.

The use of an alloy steel in the large bridge which is being built across the Mississippi River at Memphis is due entirely to Mr. Ralph Modjeski, who gave the matter a great deal of preliminary study and finally drew up the specifications under which bids were submitted. Mr. Modjeski should therefore have full credit for the pioneer work which he has done along this line.

Mr. Ralph Modjeski,* M. Am. Soc. C. E., regrets that the limited time and space do not permit him to discuss Mr. Waddell's interesting paper as fully as it deserves. One point, however, seems to him worth mentioning. Mr. Modjeski.

The author has prepared a large number of diagrams from which he proposes to determine the relative cost of "alloy" and "carbon-steel" spans of various description. It is usually impossible to determine the exact prices of carbon and alloy steel in a proposed bridge until bids

* Cons. Engr., New York, N. Y.

Mr. Modjeski. are received. Any estimates therefore made from tables are liable to be inaccurate. Besides, the designs, especially in cantilever bridges, vary to such an extent, as to general proportions and arrangement, that tables, no matter how complete, could not be applied with any degree of certainty. In order to find out definitely whether a carbon-steel or an alloy-steel design will be cheaper, Mr. Modjeski is of the opinion that the only certain method is to draw up designs on the basis of all-carbon, all-alloy, or mixed materials and to submit these designs to the manufacturers for tenders. He followed this method in securing tenders for the superstructure of the new Memphis Bridge. Specifications and plans were drawn for a carbon design and also for an alloy design with carbon floor systems. Designs were made entirely distinct and were submitted to bidders who were requested to bid either on one or the other or both of the designs.

In Mr. Modjeski's Memphis specifications, which the author of the paper quotes in part, a further provision was made to allow broader competition, namely, that the bidders were permitted to quote on an alloy of their own composition within certain limitations, provided this alloy met with the physical requirements of the specifications. This has resulted in the following bids received:

Alloy (Mayarí) design	$1,634,228.00
Carbon design	1,702,030.00

The above bids were by two different manufacturers. The unit prices per ton erected complete were:

Alloy (Mayarí) Steel in the "Alloy Design"	$115.60
Carbon Steel in the "Alloy Design"	97.20
Carbon Steel in the "Carbon Design"	88.00

The first two figures were accepted as corresponding to the lowest bids on the whole, which is represented by the first one of the two bids quoted above.

Here, then, the difference between carbon and alloy steel as quoted by the same manufacturer is 0.092 cents per lb., and the difference between the alloy steel of one manufacturer and the carbon steel of another is 1.38 cents per lb.

This illustrates further how difficult it would be to procure in advance the unit prices for certain carbon and alloy steels in order to apply the tables. This difficulty would still be more marked in smaller bridges where a larger number of manufacturers would enter into competition.

Mr. Waddell. **Mr. J. A. L. Waddell**, in closing, said that in answering Mr. Reynders' comments, the author would state that he is becoming more and more convinced that Mayarí ore will ultimately afford the solution of the problem of finding a high-alloy steel for bridgework. At present Mayarí steel is more economic than any other alloy of steel that is well estab-

lished; and there ought to be some cheap process for purifying it so as to make it more regular and of somewhat higher elastic limit—for instance, the addition of a little titanium as a scavenger. Again, the addition of a small amount of chromium or a slight increase in the percentage of manganese would be inexpensive; and either of them would certainly make the product stronger. It would not cost much to determine the greatest percentages of chromium and manganese which it is safe and perfectly satisfactory to employ for bridge metal that is to be sub-punched and reamed, or for that which is to be drilled solid. The author believes that by these suggested means a Mayarí steel of at least 60,000 lbs. elastic limit can be obtained for the manufacture of bridges of short span and of ordinarily long span, and at a price which cannot be equalled by nickel steel (unless unpurified ferro-nickel can be utilized in its manufacture) or by any other alloy of steel now known or contemplated. Mr. Waddell.

As for the obtaining of a truly high alloy of steel for very long span bridges, it seems reasonable to hope that a proper combination of vanadium with chromium, manganese, and the Mayarí product will secure an eye-bar steel of 80,000 lbs. elastic limit, a solid-drilling steel having an elastic limit of 75,000 lbs., and a sub-punching steel with an elastic limit of 70,000 lbs. Whether annealing of the plate-and-shape steel will be necessary can only be determined by experiment; but the author surmises that it will not. Of course, for the eye-bar steel that process will have to be employed; but such would be no hardship, because all eye-bars have to be annealed. As the cheapest kind of Mayarí steel is very little more expensive than carbon steel, and as it is fully forty (40) percent stronger, it seems logical to assume that in the search for a very high alloy it would be best to adopt the Mayarí product as a base and study how to improve its strength and regularity at reasonable cost.

The author disagrees fundamentally with Mr. Modjeski in his opinion that diagrams will not indicate truly the economics of various kinds of steel for bridgework, because he has proved beyond the peradventure of a doubt that they do. It is only in cases of very close results from the diagrams that actual designing need be resorted to. Of course, no diagram will cover the variations due to the idiosyncrasies of the different designers and manufacturers; but for like designing there is no doubt whatsoever about the recorded economics of weights of metal; and current pound prices for the different kinds of manufactured bridge-steel will settle the question of total cost. The author's forthcoming book on "Bridge Engineering" will demonstrate beyond all possible doubt the efficacy of diagrams for comparing weights of metal and costs of spans in the building of bridges of all kinds.

Paper No. 109

THE ECONOMICS OF THE WORLD'S SUPPLY OF COPPER.

By

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An inquiry into the economics of the world's supply of copper presents great difficulty at all times, since the data necessary for a thorough discussion of this interesting question are, in part, incomplete and, in part, altogether lacking. At the present time it presents almost insuperable difficulties. Since the principal nations of Europe are now at war with each other, ordinary relationships of production and consumption have been disrupted, and it is beyond the power of the most astute person to foresee the readjustments that will inevitably ensue when peace is restored. Even if it were practicable now to prepare a complete analysis of the economics of the world's supply of copper, it would not be worth while, since in the interval between the time of its preparation and its appearance in print extensive modifications would doubtless become necessary. I shall therefore not attempt to do more, here, than to sketch the general features of the present situation and to indicate some of the possibilities of the future.

In the preceding paper Mr. Hawks and myself have shown how, coincidently with the development of the electrical industry, the consumption of copper has increased at an extraordinarily rapid rate, and is now at the maximum in the history of the metal. This is graphically indicated by the accompanying diagram. It remains here to consider (1) whether this increase may be expected to continue; (2) whether the resources are available to permit the continued production of constantly increasing amounts of copper. As sub-topics under

(1), there are the questions: (a) whether substitutes may not be devised to take the place of copper; (b) whether changes in industry may not be brought about that will perhaps decrease the use of those materials in which copper is now used; (c) whether new uses for copper may not be devised that will increase its consumption. As sub-topics under (2), it is necessary (a) to review the known and probable resources of the world's copper deposits, and (b) to analyze the world's trade in copper in so far as it bears on the economic factors governing the exploitation of these deposits.

(1a) Probability of Substitutes for Copper.

It has already been pointed out that 60 to 70% of the present consumption of copper is absorbed in the electrical industry, the larger part of it being used in the form of wire. As set forth in the preceding article, aluminum is a strong competitor of copper for long distance transmission lines, or wherever aluminum can be stranded with a wire of steel to afford the tensile strength which the aluminum itself lacks. Aluminum is much more easily attacked than copper by corrosive agencies of the atmosphere, but as long distance transmission lines are most in use in the Rocky Mountain region, where the air is comparatively pure and dry, the importance of this factor is lessened. For telephone and telegraph lines, copper seems likely to hold its own, especially in the Eastern States (where such lines are most in use), since it is resistant to corrosion, and the smaller copper wire is less swayed by the wind and holds a smaller load of snow and ice in the occasional severe storms that are so destructive to telephone and telegraph lines. It has been suggested that steel wires coated with copper would have greater tensile strength and equal conductivity, but it is difficult to make such a coating sufficiently homogeneous and adherent so that rusting will not take place beneath it, and it is also difficult to produce compound wires at a low cost. Where small wires are necessary, as in winding dynamos and motors, electric-light cord, and such purposes, there seems no possibility that copper wire can be displaced. Much of uncertainty is injected into the situation by the potentialities which arise out of the European War. Germany pro-

duces only a small part of the copper which it uses in its manufacturing industry, importing the remainder. These imports were cut off at the beginning of the war and at the time of writing (January, 1915) the price of copper in Germany was 2½ times the price prevailing in this country. This high price must stimulate the working of low-grade deposits, but, unquestionably, it is also stimulating research to seek out methods by which other metals can be substituted for copper. The isolation of France in the Napoleonic wars gave birth to modern industrial chemistry; the isolation of Germany will undoubtedly lead to further changes in chemical engineering practice. It seems quite within the bounds of possibility that before the war is ended the German people will have learned to conduct their industries with a smaller supply of copper than they formerly required. This may result from the development of substitutes, or even from changes in industry whereby there will be less need for copper.

(1b) Possible Changes in Industry.

The tendency of the age seems to be toward the increased use of electrical machinery and devices, and, as indicated in the preceding paper, this is nearly coincident with increased use of copper. Wireless telephones do not, as yet, seem to offer much promise of coming into general use; the wireless telegraph has already developed a large field in which it does not compete with the ordinary telegraph. Therefore, neither of these devices threatens to supplant methods involving the use of copper, but they afford facilities which may tend to limit the growth of the older methods. It may be said that, for the present at least, there seems no reason to suppose that the electrical industry will demand smaller quantities of copper, while in a hundred fields of industry the ever growing use of electrical equipment calls for constantly increasing amounts of the metal. In the preceding paper Mr. Hawks and myself have indicated a variety of ways in which copper can be utilized in general industry, both in increased amounts for present use and for new uses. It is impossible at this time to predict with safety the amount of copper which is likely to be required for these purposes. An interesting discussion of the

output of copper between 1879 and 1912 appeared in the Engineering and Mining Journal, Vol. 95, p. 992 (1913). The author considers the curve of the output to be an exponential curve of the form

$$r_{n+m} = r_n \, 1.0044253^m,$$

and by projecting it to 1920, estimates an annual consumption at that date of 1,606,700 metric tons, or a little over 3,500,000,000 pounds. Heath Steele, by following a similar line of reasoning, has estimated* an annual consumption of 3,000,000,000 lbs. in 1921. It must not be forgotten that both these estimates are based on the assumption that consumption will increase at the computed rate of natural growth; in other words, they do not take into consideration possible changes in industry which may affect the demand for the metal. It has been indicated above that although there are numerous possible factors which may influence consumption, the effect of these during the next decade cannot now be forecast. It will probably be safe, however, to assume that consumption will not exceed the estimate given above, and may be considerably less.

(2a) Known and Probable Resources of Copper.

In the table below are given the ore reserves of the principal copper mines for which figures are available. I have not computed the amount of copper which can be recovered in mining, milling, and smelting, since methods of treatment are constantly being improved, and exploration, in most mines, also constantly increases the reserves. I have therefore only indicated the copper content of the known reserves, which, in most cases, may be taken as a rough index of the probable amount of copper which the mines will yield, in the light of present knowledge.

Ore Reserves of Lake Superior Copper Mines.

The following table shows the copper reserves of the principal mines of the Lake Superior district, as estimated by J. R. Finlay in a report made to the Board of State Tax Commissioners of Michigan in the summer of 1911. The figures given are the recoverable copper, not the total content. Since 1911 the recovery methods have been improved and a leaching

* Eng. & Mining Journal, Vol. 96, p. 1063 (1913).

process has been devised which is expected not only to increase the recovery from ore, but also to yield large amounts of copper from old tailing.

Mine	Recoverable copper Lbs.
Ahmeek	290,000,000
Mohawk	176,000,000
Wolverine	80,000,000
Osceola Consolidated	300,000,000
Allouez	140,000,000
Calumet & Hecla	702,000,000
Osecola amygdaloid	330,000,000
Quincy	200,000,000
Isle Royale	112,000,000
Superior	75,000,000
Baltic	110,000,000
Trimountain	16,000,000
Champion	110,000,000
Lake	40,000,000
Total, July, 1911	2,681,000,000
Deduct production to Jan. 1, 1915.	
Add est. recovery from tailing	150,000,000

It must be noted that this table includes only those mines which, in 1911, were producing copper at a profit. There are numerous other mines in this district which produce (or have produced) considerable amounts without making any profit. A decrease in working costs on an increase in the price of copper would therefore largely increase the reserves given above.

Only a few of the other copper companies have published any statement of their ore reserves, but fortunately among these are several of the largest. These are shown in the table below:

	Ore Tons	Copper %	Copper Lb.
Utah Copper	332,500,000	1.47	9,642,500,000
Ray	78,380,966	2.20	3,228,800,000
Chino	94,000,000	1.80	3,384,000,000
Inspiration Consolidated	45,000,000	2.00	1,800,000,000
Miami	20,300,000	2.45	995,000,000
"	6,000,000	2.00	240,000,000
"	17,200,000	1.21	413,000,000
Nevada Consolidated	41,020,296	1.68	1,374,180,000

Figures for the Anaconda and Phelps, Dodge companies are not available; otherwise, a fair estimate of the known copper resources of the United States might be made.

For foreign companies, even less authentic data are available. The list below gives most of that at hand:

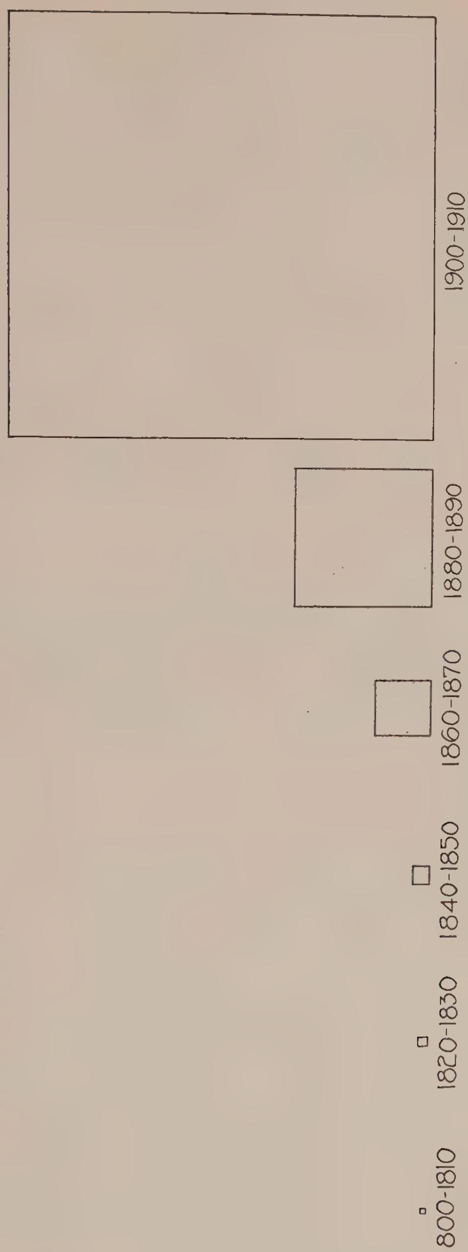
Mine	Ore Reserves Short ton	Copper Content %	Copper Content Lb.
Chile Copper Co., Chile	280,855,000	2.13	11,936,337,500
Braden Copper Co., Chile	78,000,000	2.8	4,368,000,000
Mount Morgan, Australia	3,125,000	2.55	1,572,500,000
Kyshtim Corporation, Ltd., Russia	356,000	3.00	21,360,000
Spassky Copper Mine, Ltd., Siberia	12,643	20.00	5,057,200
Mount Lyell, Tasmania	2,202,335	0.531	23,500,000
" " "	1,086,112	6.00	130,300,000

These figures are only useful as hints, since the really significant factors are the large deposits which have never been thoroughly explored. Thus the immense copper deposits of the Belgian Congo have never been appraised, since under present conditions no profit can be made, except in working the higher-grade ore. Improved facilities for mining and reduction, a new process, or higher prices for the metal may render immense quantities from these deposits available for industry. It is significant that the largest reserve in the table above, that of the Chile Copper Co., was unknown a few years ago. Large copper deposits exist in southwestern China and are only worked to a limited extent. The figures given for the two Russian companies undoubtedly represent only a modicum of the copper which that country will eventually produce. All in all, it may safely be said that the known ore reserves of copper seem to be ample for present needs, and that the needs of the future may be safely left to be cared for by exploration and investigation by future generations.

(2b) World's Copper Trade.

At the time of beginning the preparation of this paper, it was hoped that sufficiently comprehensive statistics might be secured to permit an analysis of the world's trade in copper. All

efforts to that end have proved unavailing, and the subject must therefore be dismissed with a brief summary. The United States produces more than half the world's supply of copper, Japan with the next largest output, producing less than one-seventh of the U. S. output. Of our yield about one-half is used in this country and one-half exported, chiefly to Europe, Germany having been our best customer. Japan uses a large part of its output and sells the rest in China and Europe. Much of the Australasian production goes to Europe, and the same may be said of the Mexican and Chilean copper. The copper output of Russia is absorbed in that country; in time Russia may become an exporter. Spain and Canada are also important producers of copper; the output of the other countries is small. It may be said that the general movement of copper, the world over, is toward regions of dense population, where it is put to the uses indicated in the preceding paper. Manufactures of copper, on the other hand, tend to flow outward from the densely populated regions, making their progress as far afield as the demands of trade permit. In general, it may be said that such fluctuations in the price of copper as occur in the near future will be due to the relations between demand and immediate supply, and not to any scarcity of copper ores.



World's Copper Production by Decades, 1800-1910.

DISCUSSION

Prof. Marx. **Prof. G. H. Marx,*** Mem. Am. Soc. M. E., inquired if the constituent elements of the copper, zinc and tin alloys could be recovered if the elements themselves are later preferred.

Mr. Read. **Mr. T. T. Read** replied that he could not answer that question, as his knowledge of alloys is limited. However, the scrap value of these alloys is very high if they are to be used again as alloys, and the demand for the alloys will always be high enough to use all of the scrap obtainable.

Mr. Duncan. **Mr. L. Duncan,†** Mem. Am. Soc. M. E., stated that for five years prior to the war, the Germans are said to have imported 40% in excess of the amount of copper consumed by them and to have stored it.

The last column in the table at bottom of page 510 must be reduced about 30%, since it represents copper in the ore and not the quantity of recoverable copper. The ore reserves are really much more than the amount indicated in these tables, as the mining companies drill only enough of the deposits to insure twenty or thirty years' operation of the mine. Each year more territory is drilled in order to compensate for the amount of ore removed during the year past.

Mr. Read. **Mr. T. T. Read**, in reply to Mr. Duncan's remarks, said that the point is covered in the paper, where it is stated that present recoveries are not necessarily safe guides in judging the future, and that constant increases in ore reserves are likely to more than cover the losses in milling.

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Paper No. 110

THE PLACE OF COPPER IN THE PRESENT ENGINEERING FIELD.

By

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Copper is what may be called a rejuvenated metal. As the chief constituent of brass and bronze, the principal materials of the early metal-workers' art, it was of leading importance from the dawn of history to the end of the Medieval period. Then began a period of advancement in the art of smelting iron from ores, the production of steel, and the working of the metal. Iron and steel have unique combinations of useful properties, so that their production and use attained such proportions as to almost totally eclipse copper by the beginning of the Nineteenth Century. Volta, Watt, Maxwell, Faraday, Siemens and many other scientists were then engaged upon a series of researches on electrical phenomena that were only of academic importance. Their practical application was soon perceived by such men as Morse, Bell, Edison, Brush, and in the latter half of the Nineteenth Century the telegraph, the telephone, the dynamo, the motor, the incandescent lamp and the methods for transmitting and distributing electricity had opened for copper a vast field of usefulness. In this field it is firmly intrenched, for there is at present no apparent possibility of any substitute ever being devised. It is probable that the electrical industry today consumes between 60 and 70 per cent of all the new copper produced.

How important the electrical industry is as a field for the

use of copper is shown by the following table showing the forms in which copper is sold by the copper producers:

Copper Sold by Producers in 1913.

Form	Lb.	%
Wire bars	938,362,248	58
Ingots and ingot bars	374,700,493	23
Cakes	152,442,677	9
Cathodes	130,429,587	8
Other forms	28,087,047	2
Total	1,624,022,052	100

Wire bars are cast 3 to 4 inches square and 3 to 7 feet long, weighing 150 to 500 lb. each, and, as the name implies, are used for making wire. The usual sizes sold weigh about 200 and 225 lb. each. Ingots are used for making brass and other alloys and average about 20 lb. in weight. Cakes are usually cast square, the following sizes being ordinarily sold: 14 by 17 in., weighing 200 to 250 lb. each; 18 by 18 in., weighing 200 to 400 lb. each; 20 by 20 in., weighing 300 to 500 lb. each. Special sizes, up to perhaps 2500 lb. each, are occasionally sold for special purposes, such as making locomotive fire-boxes and large copper vessels. Sheet copper in all its forms is made from cakes.

Electrical use immediately imposes a conductivity requirement which rules out everything but electrolytic and high-conductivity Lake copper. Most of the electrolytic refiners aim to average about 100.0% soft in the electrical conductivity of their output. Occasional lots may reach 101.0% and some may approach 99.0%, while 98.5% is the usual rejection limit, but it is very unusual for a refinery to ship anything for electrical use which is below 99.0%. No distinction is made between cakes, wire bars, and ingots, more than one shape often being cast from a single furnace charge, so that there is nothing to be gained by buying ordinary wire bars and cutting them up when ingots are desired.

In addition to wire, the electrical industry consumes large quantities of copper tubing, copper bars, copper strips, sheet copper, ingot copper for copper castings, and brass in all its shapes and forms; therefore, the figure of 60 to 70% of the total copper produced as being used by the electrical industry is prob-

ably nearly correct. Aron Hirsch & Sohn in "Copper Statistics for 1913" gives the distribution as shown in the following table for the copper consumed in the United States in 1913:

	Lb.	%
Electrical industry (copper wire)	400,000,000	52.1
Brass mills	220,000,000	28.7
Copper sheets	105,000,000	13.7
Miscellaneous (chiefly casting and alloys)	42,000,000	5.5
Total	767,000,000	100.00

How rapid has been the growth of the consumption of copper as a result of the development of the electrical and other industries is shown by the following curve.

USES OF COPPER.

The main claims that copper has to consideration and, in fact, to preeminence amongst the metals of commerce, so far as its special qualities are concerned, are:

(a) High electrical conductivity.
(b) Its capacity for conducting heat.
(c) Extreme ductility.
(d) Malleability.
(e) Resistance to corrosion.
(f) Alloying properties.
(g) High tenacity.
(h) High scrap value.
(i) Artistic color and lustre.

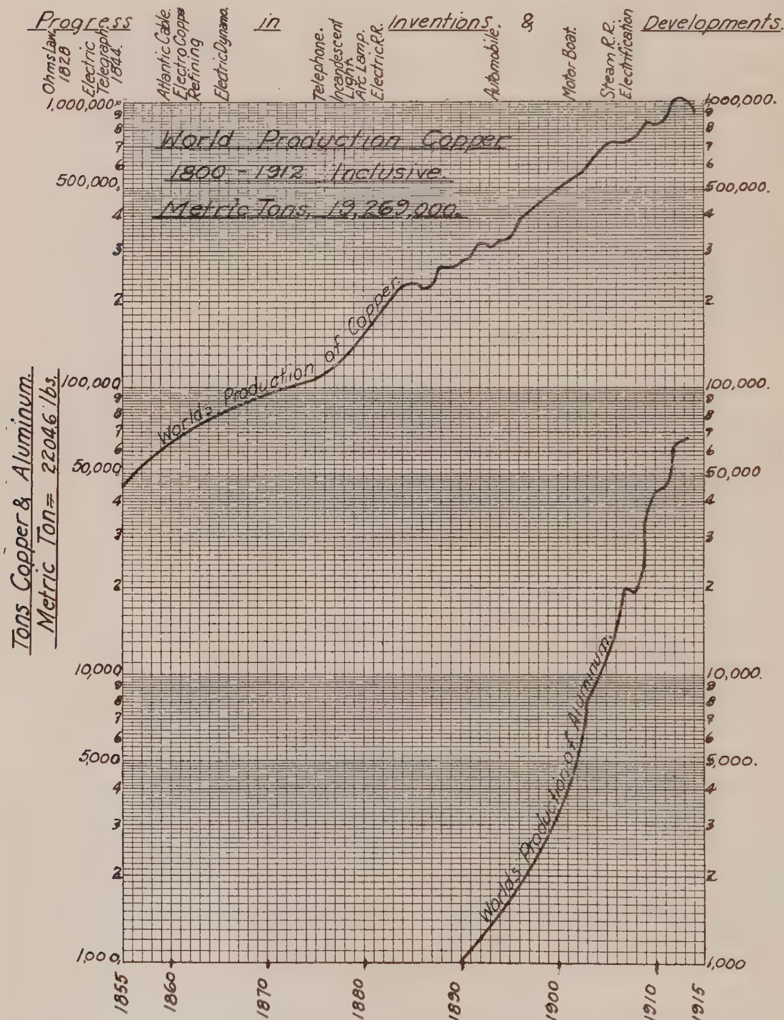
The chief use of copper, as already indicated, is as a conductor for transmitting electric currents. It is commercially the best metal we have, although substitutes, such as aluminum and iron, are used under certain conditions. A metal to be most suitable must have:

(1) A market price permitting its use on a large scale.

(2) High electrical conductivity in order that the diameter of the wire to transmit a given amount of energy may be a minimum.

(3) Good ductility so that the metal may be easily drawn into wire.

(4) High tenacity so that the wire may resist breakages from the mechanical strains to which it is subjected while being handled and used.



(5) High resistance to corrosive agents.

(6) Economic scrap value.

(7) Proper position in electro-chemical series.

None of the other metals, with the exception of silver, has sufficiently high electrical conductivity and other qualifications, except aluminum. For the same physical dimensions aluminum has about 60% of the conductivity of copper.

How and to what extent copper is sold to the various copper and brass mills, foundries, etc., has already been stated. The mills and foundries produce wire, sheets, tubes, rods, castings, plates, rivets, nails, etc., which are then sold for various uses. An incomplete list is as follows:

Wire and Bar:
- Copper
  - Electrical conductors
  - Bus-bars
  - Commutator segments
  - Copper wire for miscellaneous uses
- Brass and Bronze
  - Screens
  - Springs
  - Shoe nails
  - Pins (plain, safety and hat)
  - Small screws
  - Rivets
  - Screen cloth for paper mills
  - Lantern screens
  - Shade holders of lamps
  - Bathroom fixtures (soap-holders)
  - Buckles
  - Bird-cages, etc.

Brass Bars (Square and Rectangular):

The largest quantities are used in blanking-out nuts for bolts. This blanking is done while the metal is cold. Considerable amounts are also used for doors, partitions, etc., for buildings. Also largely used in engineering where steel or iron will not serve because of rusting.

Bronze Bars (Square and Rectangular):

Commercial bronze bars are chiefly used for doors, partitions, etc., in fine buildings; also for cemetery-vault doors. Not a great many commercial bronze bars are used in the course of a year,

but there are many uses for the special bronzes, such as Tobin bronze, manganese bronze, etc., referred to elsewhere.

Sheet Copper:

- Braziers' copper:
  - Beer stills, heavy tanks, anodes, boiler-plates.
- Plain hot-rolled:
  - Valleys, gutters, roofing, tank-linings, flashings, gaskets, floats for water-tanks, show-cases, outside and inside trim around stores.
- Plain cold-rolled annealed, tinned:
  - Drawing into pots, kettles, pans, boilers, dairy appliances.
- Cold-rolled annealed:
  - Drawing and spinning into ornamental work, kitchen utensils, percolators, gasoline tanks.
- Plain cold-rolled:
  - Skylights, cornices, conduit-pipes, table-coverings, fire-extinguishers, talking-machines, matrices, stove reservoirs for hot water, laundry machinery.
- Cold-rolled polished:
  - Reflector gas heaters, etching, copper lining for coffins, etc., hot-water heaters, hot-water boilers, bar work, pantry covering, soda-fountain sheets, engravers' copper.
- Sheet Brass:
  - Building fire-proofing
  - Cartridge-cases
  - Automobile radiators
  - Hardware specialties
  - Electrical fixtures and appliances
- Seamless Tubes:
  - Copper:
    - Hot-water heaters
    - Steam lines
    - Shrapnel shell compression-rings
    - Automobile radiator tubing
  - Tinned for dairy and cream separator appliances
  - Gasoline and oil-piping on machinery

- Steam and brine lines
- Boiler-tubes
- Distilling-apparatus
- Hose nozzles
- Electric wire-terminals

Brass and Bronze:

- Hand railings
- Plumbing goods
- Pump-cylinder linings
- Automobile radiators
- Steam and brine lines (low and high pressure)
- Bushings
- Art metal work
- Condenser tubing
- Gasoline and oil-piping on machinery
- Automobile engine intake manifolds
- Voice tubing
- Mail and small parcel delivery tubing

Brazed Tubes:

- Plumbing and plumbing fixtures
- Car window frames
- Windshield frames
- Chandeliers and gas fixtures
- Lamps
- Railings
- Bedsteads
- Coffin-handles
- Clamps for surveyors' instruments
- Pump linings
- Lightning rods—copper
- Display stands
- Piston rod covering
- Radiator tubing
- Gas burners
- Voice tubing

Perforated copper and brass sheet is used in many ways: such as centrifugal screens for sugar, pulp screens for paper mills, gas water-heater burners, false bottoms for dyeing ma-

chinery, and brass register faces for covering hot-air ducts. Sewage-disposal plants use large amounts of bronze screens, or perforated sheet. Perforated bronze sheets and screens are often used for screening coal, as the sulphur in the coal, in the presence of moisture, rapidly corrodes iron screens.

In the early days of the sheet-copper industry, the largest use of hot-rolled copper was for the purpose of sheathing wooden ships. This was known as sheathing copper. Owing to the decline in the use of wood for ship-building, the consumption of sheathing copper is now very much less than formerly. Large quantities of sheet copper were formerly used for the manufacture of bath-tubs, but this also has declined. Beer vats, stills, and coppersmiths' work consume large quantities of hot-rolled copper. The demand for cold-rolled copper is now so extensive that it has really surpassed that of hot-rolled. The great demand for cold-rolled copper which exists is because that variety of copper is flat, smooth, and stiff. Probably the largest single consumption of cold-rolled copper at present is for cornice copper. Other large uses are for boiler bottoms, kettle bottoms, etc. The largest use of hot-rolled sheet copper is probably for copper roofing.

According to German writers, the principal industrial consumers of copper are the electrical industry, the makers of brewers' and distillers' plants, shipbuilders, makers of plate (including plates for textile printing), the bronze and brass industries and the chemical dye industry. Having sketched the relative amounts and principal forms in which copper is used in engineering we will now proceed to describe its principal applications in more detail.

COPPER WIRE USED BY TELEPHONE AND TELEGRAPH COMPANIES.

The wire used in toll service by telephone companies varies from No. 8 B. w. gauge, weighing 435 lb. per mile, for lines over 800 miles long to No. 14 gauge, 102 lb. per mile, for local service. For aerial and underground cables the range is from No. 10 B. & S. gauge, 166 lb. per mile, to No. 16 gauge, 43 lb. per mile.

For subscribers' loops, No. 19 or No. 22 B. & S. gauge, 20.7 and 10.2 lb. per mile respectively, is used according to the distance from the exchange. This applies to cables; for open wire, No. 14, weighing 102 lb. per mile, is used. In the wiring of switch-board and test-boards No. 16, 42 lb. per mile, No. 18, 25.6 lb. per mile, and No. 22, 10.2 lb. per mile, are chiefly used.

Telegraph toll service.—For this No. 8 B. w. gauge, 435 lb. per mile, and No. 9, 208 lb. per mile, are used, the larger wire being employed in such important circuits as from New York to Chicago and to Minneapolis. For cables, No. 10 B. & S., 166 lb. per mile; No. 13, 83 lb; No. 14, 65 lb; and No. 16, 42 lb. are used, according to the length of the line and the character of the service, the smaller wires being used for stock-tickers, call-boxes, clocks, etc. In wiring switch-boards, repeaters, and operating tables, the sizes range from No. 14 B. & S., 65 lb., to No. 22, 10.2 lb., according to requirements.

Approximate Amount of Copper in Telephone and Telegraph Companies' Plants.

	Lb.
(1) Entire Bell Telephone Co. system.	
(a) Outside plant, consisting of open wires, underground, aerial and submarine cables	500,000,000
(b) Inside plant, consisting of switchboard and other cables	20,000,000
Total	520,000,000
(2) Western Union Telegraph Co.	
(a) Outside plant, consisting of open wires, underground, aerial and submarine cables, but not including ocean cables	120,000,000
(b) Inside plant, consisting of switchboard and interior lines	4,000,000
Total	124,000,000
Grand total for two companies	644,000,000
Postal Telegraph & Telephone Co.	40,000,000
Independent, private, special and railroad systems	50,000,000
Total	734,000,000

(These figures are approximate as of July 1, 1914.)

As the United States has about 60% of the world's telephone and telegraph mileage, 490,000,000 lb. copper may be estimated as used by all countries outside of the United States, making a grand total of the world's use of copper in this industry of 1,204,000,000 lb.

To this amount should be added the copper used in telephone and telegraph instruments, which in itself is a very considerable amount, as these instruments contain large amounts of brass. There should also be added the copper in submarine cables. There is laid today approximately 314,000 miles of submarine telegraph wire, using possibly 19,000,000 lb. of copper.

At the present time, the world's annual consumption of copper in the telegraph and telephone business amounts to approximately 85,000,000 lb.

In the telephone circuit established between New York and San Francisco there are two complete metallic circuits, utilizing 5,900,000 lb. copper. In other words, in telephoning from New York to San Francisco nearly 3,000,000 lb. copper is employed.

THE ELECTRICAL INDUSTRY.

Electric circuits: the telephone, telegraph, power transmission, and in machinery, absorb 60% of the world's output of copper, amounting in 1913 to over a billion pounds of copper. The first two have already been discussed.

The power-house of an electric system requires 4 to 10 lb. copper per horsepower. Assuming an average of 8 lb., the 12,000,000-hp. of the railway, lighting, and municipal plants of the United States listed in the census reports represents a total of 96,000,000 lb. copper. This does not include privately owned plants, which probably represent an equal amount. Electric light fixtures, switches, fuse, etc., are of brass or bronze, containing 60 to 80% copper. One firm making a specialty of manufacturing fuse and switches employs 500,000 lb. copper in its various forms, and consumes 150,000 lb. brass annually. Extensive use of copper is made in commutators, for which it has no rival, as it is the only metal which will resist the tendency toward arcing between the brush and the commutator surface

without developing hard spots on a rough surface. It quickly assumes a good polish, and, being a good conductor of heat, distributes the heat developed at the contact surface.

POWER TRANSMISSION.

The distribution of energy in industry has heretofore chiefly been accomplished by the mining of coal in the various places where it occurs, its transportation by the railroads to the points required, and the generation of energy there by combustion and steam production. Through the development of technique it has now become possible to generate energy at the point where it is available, either water-power, coal, or petroleum, and transmit the energy through conductors to the points where it is required. This permits water-power to be used at a distance from the point at which it is available, saves the transportation and distributive cost on coal, and leads to higher efficiency in the production and utilization of energy through the use of large units that are much more efficient than small ones. This method has reached the stage of development where in a system involving a direct transmission of 250 miles it is proposed to transmit 150,000 hp.

As an example, the growth of the Montana Power Co. may be briefly described. Fifteen years ago a few thousand horsepower were developed at the Madison River and transmitted 61 miles to Butte, Montana. The company now has in operation 46 power-transmission routes, requiring 4400 miles of wire and 4,500,000 lb. copper; also, 2250 miles of telephone wire, using 450,000 lb. copper. To provide for increasing business, 2,600,000 lb. more copper will soon be required. The 3-phase transmission line now in use would suffice to build a line from San Francisco to New York, and to construct a telephone circuit over the same distance. When its present plans are completed the company will be using 10,000,000 lb. copper in its transmission lines, switch houses and power houses. If it were possible to include the copper employed by the consumers of the current supplied by this company the amount given above would doubtless be increased several times. The copper used by the two principal consumers is listed elsewhere.

COPPER WIRE AND CABLE—GENERAL.

Copper wire and cable is made in sizes ranging from a fine wire weighing a few ounces per mile, up to cables weighing 32,900 lb. per mile. It is estimated that 250,000,000 ft. of copper wire, weighing between 12½ and 68 lb. per mile is used annually in the United States. In addition to the uses already discussed much wire is employed in call systems, fire-alarm systems and burglary systems for banks and private residences. These devices not only require much wire but are also considerable consumers of brass for their annunciators and other accessories. Much wire is employed in electric firing of blasting charges, and a good deal of copper is consumed in the shells of blasting caps. A considerable amount of copper wire is used in lightning rods.

In Europe and in districts where heavy loads of snow and ice are likely to cling to exposed wires, silicon bronze is largely used for this purpose. Silicon bronze is valuable because of its great strength combined with high conductivity. It finds considerable use as trolley wire and span wire because of these qualities and, in addition, it does not corrode under the action of the sulphurous gases of combustion with which the air is constantly laden.

Steel wire surrounded by a coating of copper of any desired thickness, known to the trade as "Colonial copper clad", "Bimetallic", "Duplex wire", and "Monnot wire", has become of importance. Ordinarily the copper area is about 1/3 that of the steel. The wire is made by suspending a steel ingot in a mold and casting copper around it. By proper procedure perfect adhesion of the copper to the steel is secured; the billet is then rolled into rods and drawn into wire. The drawing must be done with care and costs more than ordinary wire-drawing. The product has a high elastic limit, high tensile strength, high modulus of elasticity, low coefficient of expansion, and high resistance to corrosion. The coefficient of expansion is only 2/3 that of copper and ½ that of aluminum, thus small variations in sag with great variation of temperature can be secured. It is lighter than copper wire and is used for long spans, such as river crossings, and as a ground wire for lightning protection. It is often used for telephone wire on power-transmission circuits, as it may

be placed on the transmission towers, whereas a separate line would have to be constructed for copper wire. It is also largely used in municipal, fire, and police telegraphs, for railways in their signal systems, where strength is desirable, and for bond or tie-wires in place of galvanized wire.

For telephone lines this wire is not likely to entirely displace copper. In some places for open-line distribution it is the best material. For subscribers' connections copper-clad wire can be used, having the same strength as copper but less than half the weight, and because of its smaller size supporting easily a greater weight of sleet. For telephone lines up to 200 miles in length No. 9 B. & S. gauge copper-clad gives satisfactory service, but for long toll lines and for small wiring, cable copper is the best material.

COPPER IN ELECTRICAL TRACTION.

The street railways are large copper consumers; the ordinary city electric car will contain 1000 to 2500 lb. copper, and some of the large interurban cars will have nearly 4000 lb. As an illustration, take a street railway system in a city of about 150,000 people; the copper used is approximately as follows:

	Lb.
Feeder system	2,433,000
Overhead trolley	528,000
Track-bonding and special work	225,000
Car equipment	510,000
Power-house	46,000
Total	3,742,000
Number of cars operated	550
Total mileage based on single-track miles	176
Capacity of generating station	6,800 kw.
Average copper per mile of track	21,400 lb.

Many electric roads will show a smaller figure, and some in large cities, such as New York, Boston, Chicago, and San Francisco, will show a higher figure, but assuming a figure of 15,000 lb. copper per mile and considering that there are approximately 45,000 miles of electric railways in the United States, approxi-

mately 675,000,000 lb. copper is used in the street-railway business in the United States.

A considerable amount of new copper is used annually to maintain the electrical equipment on electric railways; windings burn out or the insulation is damaged; commutators wear, controller and circuit-breaker parts become damaged; the trolley wire breaks or partly wears out. All of the copper in these parts is sold for scrap and new copper purchased. A 10-car train in the New York City subways will contain approximately 30,000 lb. copper, used as copper wires and bars in the electrical equipment and as an alloy in the many brass parts.

Steam Railroad Electrification in the United States.

New York Central R. R., suburban district at New York:	
Miles electrified, single-track basis	255
Copper in generating and substation equipment, cars and locomotives, trolley, bonding, and distribution system; total copper, lb.	9,100,000
Amount of copper used per mile of track, lb.	35,600
New York, New Haven & Hartford R. R. New York to New Haven electrification:	
Miles electrified, single-track basis	550
Copper in generating and sub-station equipment, cars and locomotives, transmission and distribution system; total copper used, lb.	4,670,000
Amount of copper used per mile of track, lb.	8,200
Norfolk & Western Railway Co., Elkhorn Grade—Bluefield electrification:	
	Lb.
Transmission	175,000
Distribution, including trolley, feeders, etc.	377,000
Track-bonding	87,000
Ground-plates	55,000
Locomotives	300,000
Transformers	70,000
Generators	56,000
Switchboards, wiring, and miscellaneous	25,000
	1,145,000
Total single-track mileage, miles	90
Amount of copper used per mile of track, lb.	12,700

West Jersey & Sea Shore R. R. Co.,
Camden to Atlantic City:

	Lb.
Power-house and sub-station	310,000
Electrical equipment of cars	245,000
Transmission wires, trolley wires, feeder cables and track bonds	1,356,000
	1,911,000
Total single-track mileage, miles	150
Amount of copper used per mile of track, lb.	12,700

Butte, Anaconda & Pacific Railway Co.,
Butte-Anaconda, Montana:

	Lb.
Sub-stations	29,750
Locomotives	173,700
Distributing system, trolley, feeder cable and track bonds	916,382
	1,119,832
Total single-track mileage, miles	95
Amount of copper used per mile, lb.	11,800

Chicago, Milwaukee & St. Paul Railway,
Main Line from Harlowton, Mont., to Avery, Idaho:

Approximate main line distance, miles	440
(At present work is being rapidly pushed to complete the eastern half of this work, that is, from Deer Lodge, Mont., to Harlowton, Mont.)	
Miles electrified, Deer Lodge-Harlowton, single-track basis	321
Copper in sub-station equipment, locomotives, transmission and distribution systems, lb.	7,550,000
In addition, the signal system will use approximately, lb.	250,000
Additional telephone facilities in connection with this work will require, approximately, lb.	110,000
Total copper, lb.	7,910,000
Amount of copper used per mile of track, lb.	24,600

As soon as the Deer Lodge-Harlowton work is completed, the line from Deer Lodge to Avery, Idaho, will be finished; this will require approximately the same amount of copper as the former. This system will be supplied with power from the Montana Power Co.'s lines and will be fed from a high-tension transmission net-work of wires involving the use of 7,000,000 lb. copper.

From the above, and considering improvements in the way of voltages, 10,000-25,000 lb. copper per mile may be considered to be an average minimum figure for future work. If all the railroads in the United States were electrically operated it would require the world's entire output of copper for two years at the present rate of production, the mileage being approximately 330,000 miles on single-track basis.

Steam and electric railroads are also large users of copper and copper-base alloys for other purposes besides electrification. For instance, in normal times there is consumed in the United States annually 7,000,000 to 8,000,000 lb. copper in air-brake apparatus alone. Electric signalling also requires many thousands of pounds of copper. The amount of copper used in automatic block signalling varies all the way from 50 lb. per mile of track, which would be what is known as wireless control on multiple-track steam railways, up to 1000 or 1500 lb. per mile of track for signalling in such cases as the New York subways. In the interlocking part of signalling the amount of copper used varies all the way from 25 lb. to 200 lb. per function, depending on the kind of signalling used. By function is meant a switch or signal.

An up-to-date steam locomotive uses considerable copper. On American-designed locomotives the use of pure copper is confined almost exclusively to copper pipes and ferrules. In foreign practice copper is also used for firebox-sheets, tubes, and stays. Copper pipe is used for injector steam-pipes and practically all oil-piping and steam gauge and lubricator steam-pipes. When injector delivery-pipes are used inside of boilers they are also made of copper. An analysis made of the copper used in a representative locomotive is about as follows, in pounds: Copper pipe and ferrules, 150 lb.; driving journal bearings, 1,600; engine-truck journal bearings, 70; trailing-truck journal bearings, 140; main and side-rod bearings, 550; motion-work bushings, 100; dry-pipe sleeves, 85; steam turret, 230; miscellaneous castings, 475; bell, 125; injectors and lubricators, 180; cocks and valves, 50; safety valves, 75; electric headlight (when used), 10. This list represents about 3000 lb. copper.

During the period from 1900 to 1914 inclusive 68,000 locomotives were built in the United States and Canada, and assum-

ing only 2000 lb. copper was used in each locomotive, we have a total of 136,000,000 lb., an average of 9,000,000 lb. per year. Many of the brass parts are subject to wear and their replacement requires many more pounds of new copper annually.

Practically all freight cars, passenger cars, and locomotive tenders in the United States and Canada are equipped with M. C. B. (Master Car Builders) journal bearings of one of the following sizes: 4¼ by 8 in., 5 by 9 in. and 5½ by 10 in. There are a few smaller sizes in use and a few larger. An average of the different compositions used will be about: copper 77%, tin 8%, lead 15%. The bearings are bored where they come in contact with the journal and are lined with a composition about as follows: lead 85%, antimony 10%, tin 5%. The thickness of the lining varies with different roads.

Capacity of car, lb.	60,000	80,000	100,000
Size of journal, in.	4¼ by 8	5 by 9	5½ by 10
Bored new, lb.	11.5	17.6	23
Complete with lining, lb.	13	19.5	24.6
Weight of scrap (approximately), lb.	8	12	15

The scrap weight varies considerably, some having the same weight as a newly-bored bearing. In other words, the wear will just about equal the weight of the lead lining. For passenger-car work, the latter condition would probably obtain, while on freight cars the bearings are worn very much more. The worn bearings are remelted to produce new bearings, but new metal must also be used, the custom being to use perhaps 1/3 scrap and 2/3 new metal.

The average weight of copper in the 60,000-, 80,000- and 100,000-lb. capacity cars is about 120 lb., allowing for some other small uses of copper. An ordinary passenger car will contain from 150 to 800 lb. of copper. The modern passenger car, equipped with axle-driven lighting system, six-wheel trucks and the many brass and bronze fittings, may approximate 800 lb. of copper. A representative Pullman sleeping car with axle-driven lighting generators will use approximately 1400 lb. of copper.

During the period from 1900-1914 inclusive 2,236,192 freight cars and 45,235 passenger cars were built in the United States.

On this basis 268,429,440 lb. copper was used in freight cars and approximately 148,070,560 lb. in passenger, Pullman, and private cars and locomotives, making a total of 416,500,000 lb. in round numbers, or an average, during the period 1900-1914 inclusive, of 27,700,000 lb. annually in the United States and part of the Canadian and export trade.

It is estimated that fully 100,000,000 lb. of bearing-metal is melted annually in the United States to take care of the demand for new journals and to maintain those worn out. It is difficult to determine just how much new copper is used annually, but it is probably around 50,000,000 lb.

ELECTRIC LIGHTING INDUSTRY.

This is an even larger field for the use of copper than the electric railway industry, as the mileage of wires and cables is greater, and power-houses more numerous. We will not attempt here to itemize the copper used in electric lighting, as the necessary data are not available.

AUTOMOBILE INDUSTRY.

A motor-car, according to its size, will contain 10 to 200 lb. copper, employed in the construction of some 25 different parts of the equipment. During 1914 approximately 515,000 motor-cars were built in the United States. Assuming that 75 to 100 lb. copper was used in each, there results an annual consumption of 40,000,000 to 50,000,000 lb., including the scrap and repair parts. Of this approximately 10,000,000 lb. is used in radiators, in almost all cases in alloy form as brass. On January 1, 1915, there were 1,754,750 cars and trucks in the United States, and an appreciable amount of copper is consumed annually in their maintenance.

BOAT AND SHIP BUILDING.

The motor-boat industry is growing rapidly and probably consumes one-half to two-thirds as much copper as the automobile business. In a motor-boat, copper and its alloys are used in

nails, burrs, and bolts for fastening the parts together, for sheathing the hull, and for gasoline and air tanks. The latter are used to make the boats non-capsizeable and non-sinkable. Copper piping is used to connect the gasoline tanks with the engines, as it does not rust, is flexible, and the absence of joints tends to prevent leaks. The deck fittings are made of yellow brass, the rudder and propeller-shaft of Tobin bronze, the tiller-ropes of phosphor-bronze, and the stern bands of brass. The shackles, hangers, and hanger-bolts of yacht tenders are made of Tobin bronze. In some large vessels, especially those built for the Government, all the piping is required to be of brass. A 45-foot cruiser boat, for example, will have approximately the following amount of copper (in pounds): gasoline tank, 200; water-tank, tin-lined, 90; shaft (Tobin bronze), 150; rudder (Tobin bronze), 50; steering-gear, 45; brass deck-rails, 200 to 280; awning fittings, stanchions, 75; deck fittings, clocks, cleats, etc., 35; if the boat is copper sheathed, 250; engine, 30; galley, 20; nails, rivets, stem, etc., 30; a total of 1255 lb.

One prominent motor-boat engine manufacturer, in commenting on this situation, states as follows:

> "We use a great deal of copper in the construction of our engines, but as we are building so many different sizes it is rather difficult to estimate the exact amount of copper used. The crank cases are made of an aluminum alloy which contains 7% copper. The cam-shaft bearings and main bearings are made of bronze which contains 80% copper. This also applies to the cylinder covers, as well as valve tapper-guides and water pump. We should judge that the average motor-boat engine has about 35 lb. copper, and we turn out close to 3000 motors a year."

The modern battleship uses about 1,000,000 lb. of composition castings (copper alloy), about 500,000 lb. of sheet and tube copper and about 250,000 lb. copper in electric cables, motors, switchboards, appliances, etc. One of the recent superdreadnoughts used about 1,125,000 lb. of composition castings. Copper is used for main and small auxiliary exhaust piping, also for sea-pipe and water-pipe. Brass tubes are used in the condensers. The copper in the surface condenser and auxiliaries will amount

to as much as 100,000 lb. in a modern battleship. Composition castings are used for valves and fittings throughout the vessel. The propellers are generally made of bronze, and in a large war vessel the total metal in the propellers amounts to 40,000 to 50,000 lb. All the dresser tops for the pantries and galleys are covered with sheet copper. Blades for the turbine engines are all made of copper alloy. Turbine engines are rapidly being adapted both for propelling a boat and operating electrical generating apparatus. A modern battleship is literally threaded with a maze of wires for telephone, telegraph, signal and electric-lighting systems. A recently constructed battleship has 40 signalling systems. This same battleship is also equipped with 3000 electric lights, 4000 rated hp. in electric motors, using approximately 76 miles of cable for the various wires.

ALLOYS BASED ON COPPER.

Brass is a copper-zinc alloy, with copper, the chief constituent, varying between 55 and 95%. Bronze is a copper-tin alloy, the copper constituting 80 to 90% of the whole. There are hundreds of alloys of which copper is a component, many of them having established trade names. Brass is one of the most useful metallic substances. There are some 3600 plants in the United States melting brass and bronze, 1000 of them melting non-ferrous alloys exclusively.

For a full discussion of the alloys based on copper, reference should be made to the books on that subject; only one or two recently become of importance will be mentioned here.

Pivot Disc Metal is perhaps the strongest bronze, both under tension and compression, made. It is also very hard, having a Brinell hardness of No. 270. The discs for the Emergency Dams of the Panama Canal were made from this metal. Government tests of pivot-discs and gear-wheels, Panama Canal, were as follows:

Tensile strength	122,000 lb. per sq. in.
Elastic limit under tension	89,000 lb. per sq. in.
Elastic limit under compression	58,000 lb. per sq. in.

Permanent set at 100,000 lb. per sq. in. Compression — 0.014 in.

These are remarkable figures. Each revolving pivot disc, 43 in. diameter, supports 7,000,000 lb. An especially interesting manganese bronze is Parson's manganese bronze. It is very exclusively used in ship propellers and is used by the various war vessels and commercial ships. The United States, British and German governments used hundreds of tons of this metal. The propellers on a modern battleship, as mentioned elsewhere, will weigh 40,000 to 50,000 lb., and the copper content will be from 25,000 to 30,000 lb. Automobile and motor boat manufacturers use thousands of pounds of this metal. Parson's manganese bronze is easily machined and is absolutely non-corrosive in either fresh or salt water. Bronze gears and bearings are used in very large amounts in the various trades. Other alloys, such as Monel metal, nickel-copper, aluminum-copper and German silver are used for a variety of special purposes.

Every modern engine has brass grease and oil-cups, and many brass, gun-metal, bronze or composition copper bearings. Copper and brass boiler tubes are almost entirely used in foreign practice in locomotive construction. An extensive use for brass tubing is in the construction of surface condensers for steam. For each horsepower of capacity of the steam engine 1½ to 2 lb. copper is used in the condenser; thus a 1000-hp. engine operating with a surface condenser would employ 1500 to 2000 lb. copper in the condenser. One of the largest generating plants in this country, with a capacity of 400,000 hp., uses 750,000 lb. copper in its condensers and auxiliary apparatus. In addition the tubes must be replaced every few years, the service ranging from 1 year to 10, depending upon the character of the condensing water.

The average composition of valves, cocks, and faucets, of the best grade is: Cu, 83%; Zn, 10%; Pb, 4%; Sn, 3%. Most valves are made of brass or bronze and probably 10,000,000 to 15,000,000 lb. copper is used annually for this purpose. In the construction of the additional water supply of New York City 3,000,000 lb. of manganese bronze was used in the construction of the valves, stems, etc.

Tons of wire cloth are used every week in paper mills, in the form of Fourdrinier cloth, upon which the liquid pulp is run, the water draining through the meshes. The wire is made from

a mixture of copper, zinc, and tin, containing 80% copper, and the cloth has a comparatively short life, being then only useful for scrap. Pulp and paper mills also use copper vats and rolls; the vacuum-pans of sugar factories and refineries are made of copper, as are the worms and stills of distilleries, and the kettles in breweries. Copper kettles are also used in textile industries for holding the dye, and copper rolls are used for printing. In the confectionery business copper kettles are used for steaming, also cooking candy and syrup, and many copper funnels are used.

HEATING AND COOKING DEVICES.

The important factors in the making of these are

1. Thermal conductivity
2. Ductility
3. Length of service
4. Pleasing color

The relative thermal conductivity of the various metals in calories per centimeter per second are as follows:

Silver	0.993	Tin	0.155
Copper	0.924	Iron	0.147
Aluminum	0.502	Nickel	0.141

The value of copper from the thermal standpoint is well shown in the table. Being ductile it is easily worked into shape and its strength is increased by hammering.

Copper-jacketed steam kettles have been used for food products for over 70 years. Kettles are made to stand from 30 to 200 lb. steam pressure; copper is best for this purpose because of its strength and also because of its quick absorption of heat, especially for materials that would be injured by slow boiling. In the case of food products containing acids that would attack the copper, the inside of the kettle is tinned with a ⅛-in. coating; in some instances the kettles are silver-plated inside. These kettles are also used by photographic material makers in making the emulsions for photographic paper.

The following industries use copper-jacketed kettles in large quantities: All food products manufacturers, such as candy

manufacturers, syrup manufacturers, catsup or tomato pulp manufacturers, soup manufacturers, preservers of fruits, extracts and fruit syrups for soda fountain use, pharmaceutical chemists, cracker manufacturers. All heavy chemical manufacturers use a great deal of copper; also dyeing plants and color manufacturers.

Vacuum-pans, for materials that would be injured by being boiled at too high a temperature, are made of copper. All experimental laboratories use copper kettles for research work, and many chefs use bare copper for cooking kettles, claiming that the original flavor and color are better retained. Such kettles have given 50 to 60 years of service, showing the long life of the metal. Wherever cooking is carried on extensively, such as in large hotels, restaurants, etc., copper is largely used for kettles of all descriptions, hot water heaters, pails, casseroles, etc. One of the large firms making a specialty of high-grade hotel and restaurant goods, coffee, tea and hot-water urns, urn stands, cup and plate warmers, bar supplies and sundries, brass and nickel-plated cuspidors, etc., states that in normal times it consumes approximately 1,000,000 lb. copper annually—this being only one of the many coppersmiths manufacturing this kind of apparatus.

The domestic uses of copper and brass are numerous and varied. One prominent firm states that it consumes annually 60,000 lb. copper and brass, combined, in the manufacture of brass bedsteads and brass-trimmed bedsteads. Brass rods are used in millions of homes for stair carpets and for the suspension of portieres and curtains. Brass or copper lamps for burning kerosene oil are more durable than those of glass or pottery.

Sheet copper is used for lining sinks and dumb-waiters in the highest class of construction.

Copper is extensively used for surgical and medical sterilizers, largely in sheet form. One firm making a specialty of this line of business consumes approximately 50,000 lb. copper annually and nearly the same amount of brass.

Copper is used extensively in the construction of dairy machinery, in the form of tinned-copper sheets. One prominent firm states that it uses approximately 60,000 lb. of copper and tubes annually. Composition copper castings are used for shaft-

bushings and bearings, also special nuts which would be subjected to rust; also in worm-wheels which drive high-speed worm screws. There is also a considerable quantity of copper-bearing steel used, principally in cream-separator machinery.

BUILDING CONSTRUCTION.

The use of copper and its alloys for building construction is extensive and increasing. The principal use is for roofing, where its freedom from atmospheric attack gives it a long life, while the green color it soon assumes is highly artistic. Roofing in use in Faneuil Hall, Boston, removed after 95 years' service was found to be in condition to last 25 years more. The cost of laying a copper roof is approximately the same as any other, the increased cost of construction being due to the cost of the material. This is more than counterbalanced by its longer life in buildings of permanent character.

A less expensive roofing of resistant qualities is made of sheet steel containing 4 to 5 lb. copper per ton. The American Sheet & Tin Plate Co., advertises that in 1914 it sold 105,072 tons of this material, containing 500,000 lb. copper. Such sheets are, however, used for many other purposes than roofing. The resistance of copper-bearing steel to corrosion is leading to an increasing use for sidings, fencing, screens, etc.

Copper is used largely for cornices, the same considerations applying as to roofing. Copper and brass sheets are used for outside walls and facings, also for sheathing exposed interior woodwork, such as baseboards, window-sashes, and doors. A recent form of fire-proofing consists in drawing a 26- or 28-gauge sheet over a wood core.

Thousands of pounds of copper are used yearly in the making of window-screens that will not deteriorate in moist climates. These are generally made of some form of bronze wire. Bronze doors and gates for public buildings lend themselves to the exercise of art and have almost perpetual life. Bronze is also suitable for grille work, for the same reason.

As an example of the amount of copper used in building construction the Equitable building, New York, may be cited.

This is the largest office building in the world, and, together with the land, costs nearly $30,000,000. Copper is used in over 40 different kinds of material and equipment, ranging from floor-plates to signal systems. In all, 500,000 lb. copper, 200,000 lb. brass, and 350,000 lb. bronze is thus employed. This does not include the copper used in the kitchens of the clubs in the building, nor in the equipment used by tenants. Probably over 1,000,000 lb. copper is used in the building.

In the construction of the Grand Central terminal station 1,144,000 lb. copper was used in 43 kinds of equipment for the buildings; 108,900 lb. in 15 kinds for the service plant; 456,600 lb. in 8 kinds for sub-stations 1 and 1-A; 504,800 lb. in 16 kinds for the yard; 476,200 lb. in 10 kinds for electrification, signals, and interlocking; and 27,500 lb. in 5 kinds for the construction department, a total of 2,718,000 lb.

HARDWARE.

Copper and its alloys are used for hardware wherever it is necessary to protect the articles against rust, where a protection or pleasing finish is desired, or where it is necessary to stamp or spin the material. Over 150 kinds of hardware in which copper is employed might be listed. This business consumes annually 6,000,000 to 8,000,000 lb. copper in this country. The most rapid increase is due to the growing demand for automobile fittings and electrical sundries.

COINAGE.

Copper and its alloys have been used for coins since the dawn of history. In the United States silver and gold coins contain 10% copper; 5-cent pieces contain 75% copper and 25% nickel; 1-cent pieces contain 95% copper and 5% tin and zinc. During 1914, 960,000 lb. copper was used in the United States for coinage purposes. Assuming the same ratio between the copper used and the value of the coins produced indicates that a total of 126,000,000 lb. copper has been used for this purpose since coinage was first begun in this country.

MISCELLANEOUS USES.

Much copper is used annually as engravers' sheets, much is also used in jewelry; 22-k gold contains 1/12 of its weight of copper, 6-k gold is 7/12 copper. Sterling silver contains 7½% copper and platers' metal 30 to 35%.

Almost the whole works of a modern clock are made of brass; the same is true of the cheaper variety of watches. Instruments of precision for scientific work, such as microscopes, telescopes, surveyors' and draughting instruments, etc., are made largely of brass. Brass fittings are used exclusively on trunks and bags; it is estimated that 1,000,000 lb. copper is used annually for this purpose.

Copper, brass, and bronze signs are seen everywhere. Copper-lead, brass foil, and bronze paint are used by sign painters. In offices, brass is used for a great variety of purposes, ranging from brass clips and paper fasteners, and the metal tips of lead pencils, to the working parts of registers and adding machines. In the making of the latter 10,000,000 lb. copper is used annually. In making pins 5,000,000 lb. is used yearly, and 500,000 lb. is annually employed in making shoes, in the form of eyelets, hooks, nails, and screw-wire.

In fire-extinguishing apparatus 2,000,000 to 3,000,000 lb. copper is used yearly. Copper is largely used in the fittings of bars and soda fountains, both as brass and in nickel-plated German silver. Much brass is used in the trimming of harness; copper and bronze are used for making bull rings, and as one firm alone reports that it uses 50,000 lb. copper annually for the latter purpose the importance of these apparently minor uses is greater than might be supposed. Such articles as cigarette cases, smokers' equipment, pocket-books, chatelaines, fittings of umbrellas and canes, buttons, and wire brushes consume in the aggregate much copper. A considerable amount of copper, possibly 500,000 lb. is used annually in the making of cans. Copper and its alloys are increasingly used for mortuary purposes. The National Casket Co. used 16,000 lb. bronze in 1914 for caskets. Copper is used for lining vaults. Both these uses are new and likely to grow rapidly.

Copper is used in chemicals to a large extent. Thousands of pounds of copper are used annually as an insecticide, especially in Europe, for combatting the *phylloxera*. The sulphate is also used in electrotyping, and electroplating, in primary battery construction, in textile factories, in commercial industries, and in other ways too numerous to mention. In making copper wire and sheets a small amount of the black oxide is formed. This is, in part, sold to second-hand dealers, but possibly as much as 100,000 lb. per month is ground and used for making ''Marine Paint''.

AMMUNITION.

The amount of copper used for this purpose, chiefly as brass, is astonishing. Cartridge cases and shells are made from a brass containing 70% copper. Pure copper cannot be used for this purpose, as it is not resilient enough to spring back from the cartridge chamber after firing, but would be forced so tightly against them that the empty cartridge could not be extracted. The cost of a cartridge case for the U. S. Army is 1½c, the cost of the loaded cartridge 2½c. It is therefore economical to reload the cartridges and they must be carefully made in order not to be damaged in firing. The specifications for U. S. Army cartridges require them to stand 20 re-loadings. The ordinary army cartridge contains 23,000 to 28,000 lb. copper in a million cartridges, according to the make of rifle. A 3-in. rapid fire gun uses a cartridge weighing 1 lb. 5 oz., and a 6-in. gun one weighing 28 lb. 13½ oz. All shrapnel shells require a copper compression ring to enable them to follow the rifling in the guns; these rings will average 2½ to 3 lb. in weight. A new type of bullet being used in Europe is made of brass containing 90% copper. More common is the antimonial lead bullet with a cupro-nickel jacket, usually 85% copper and 15% nickel. This jacket is better than steel, which wears out the rifling. The ordinary shot-gun shell is made of paper with a brass end, and the consumption of copper in making these is large. The primers of all cartridges are chiefly copper. It is estimated that the normal consumption of copper by the ammunition makers of the U. S. is 20,000,000 lb. At present it is, of course, much larger.

CONCLUSION.

To cite all the uses of copper in industry would lead to almost endless detail and touch on almost every phase of life. It is hoped that the more essential features have been made clear. It should be added that in the preparation of this paper the work of collecting data and preparing estimates has almost wholly fallen upon Mr. Hawks, the outlining of the scope of the paper, revision, and the final preparation of the manuscript has been the work of Mr. Read.

DISCUSSION

Mr. Lindemann.

Mr. W. C. Lindemann,* Assoc. Mem. Am. Soc. M. E., remarked that it is interesting to note that the price of copper dropped during the maximum demand of last May, showing that American smelters are fully able to supply a very large demand. Consequently, sky-rocket effects in the market are due largely to artificial methods of manipulation.

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Paper No. 111

ALLOYS AND THEIR USE IN ENGINEERING CONSTRUCTION.

By

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Among the more important materials of construction are those which consist of mixtures of metals which possess more or less widely different properties from the constituents of which they are made. Such mixtures are termed alloys, and it is the purpose of this paper to point out some of the facts, a knowledge of which is essential to the proper selection of the material best suited for the specific purpose. This discussion will be confined to the properties which influence such selection and will not attempt to explain them.

The most extensively employed group of alloys is that of which copper is the base, and to them this paper will be more particularly devoted. This group comprises two principal series; one of these consisting of copper and zinc, forming the brass series; the other consisting of copper and tin, forming the bronze series. Owing to the fact that tin is sometimes added to brass and zinc, likewise to bronze, these terms are not always strictly definitive. Moreover, because of the possession of characteristics somewhat similar to the bronzes, certain mixtures which are basically brass are commonly designated by the name of bronze. Such usually carry, also, more or less distinctive appellations in addition.

While there are important uses for which alloys composed wholly of copper and zinc, or copper and tin, are most advantageously employed, there are other uses to meet which one or more other constituents are added to the mixture. While both the brasses and bronzes have been employed since the earliest times, the properties they possess are governed by many fac-

tors, the effects of which are not entirely understood. As a result, the processes by which they are prepared for use are still largely matters of art rather than of science. Enough scientific work has been done in regard to them, however, so that the laws which mainly govern their properties can be approximated.

Of the two series above mentioned, the brasses are in extent and variety of uses by far the more important. They cover a range from about 55 per cent copper and 45 per cent zinc up to pure copper, and over this range exhibit a wide

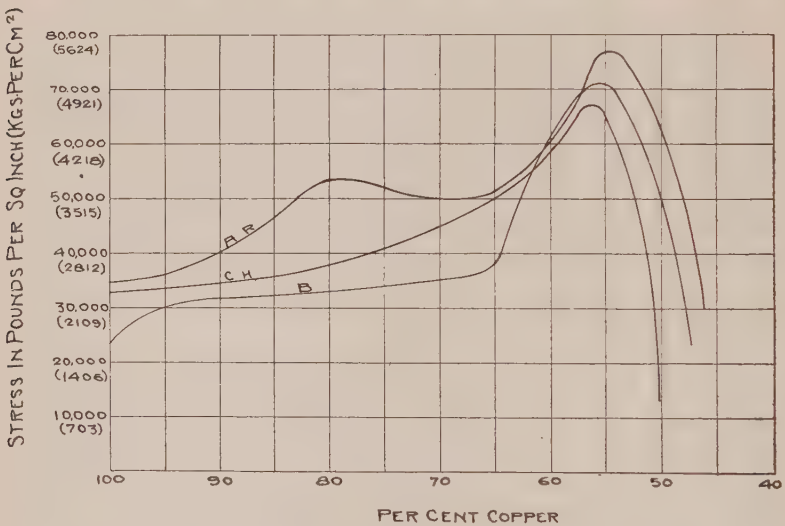

Fig. 1. Curves Showing Tensile Strengths of Copper-Zinc Alloys. AR as determined by the Alloys Research Committee; CH as determined by Charpy; B as determined by Bancroft and Lohr.

variety of qualities. The chief characteristic of this series, as also of the bronzes, is its resistance to the corrosive action of agencies which rapidly destroy iron and steel.

The brasses are employed both in the form of castings, made by pouring the molten material into sand moulds, and in the wrought state; in the latter case, being cast, in iron moulds, into bars or billets and from there, by hot or cold processes, wrought into finished or semi-finished forms.

The physical properties of the brasses vary over a wide

range according to the relative percentages of copper and zinc. Several investigations to determine these variations have been made (1).* Some of the results obtained are shown in Figs. 1 and 2. They do not agree closely with each other, undoubtedly due to some or all of the causes for variation to be later enumerated, the extent of which these curves well illustrate. Bancroft and Lohr's observations were made on cast test pieces; Charpy's and those of the Alloys Research Committee, on wrought material. From an engineering standpoint, they are particularly lacking, in that the elastic limits are not given.

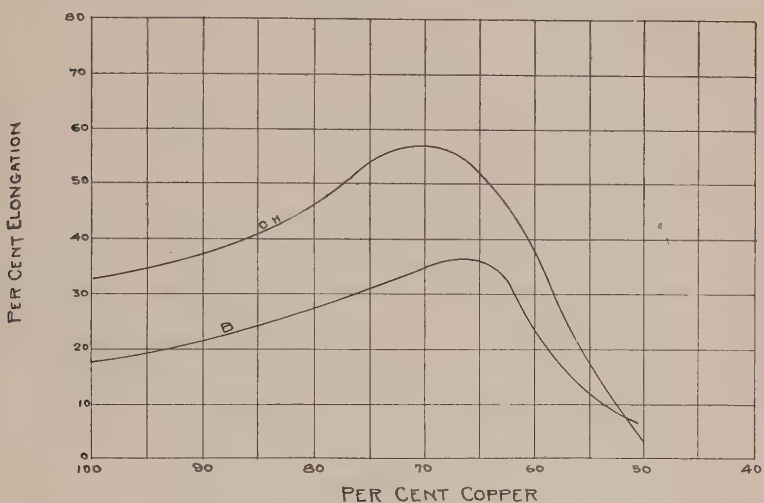

Fig. 2. Curves Showing Elongation of Copper-Zinc Alloys Corresponding to Tensile Strength in Fig. 1.

Because the ratio of the elastic limit to the tensile strength varies with different proportions of the ingredients, this information is necessary for correct design. The shape of the elastic curve is such that this value can only be determined by the use of refined apparatus.

Much more consistent data as to the properties normal to the copper-zinc mixtures are obtained by using test pieces made from material which has been given a large amount of

* The figures in parentheses refer to the numbered citations in the appended bibliography.

cold working and then annealed than is the case where cast test pieces are employed. Fig. 3 shows the properties of the series as determined, by the writer, from annealed wrought specimens made of copper and zinc selected for purity, but cast, cold rolled and annealed by commercial methods. These curves may be considered as representing the properties normal to the varying proportions of the two main ingredients. They are affected, however, to a marked degree by many other influences. Among these are variations in the temperature at which the constituents are mixed, the temperature at which

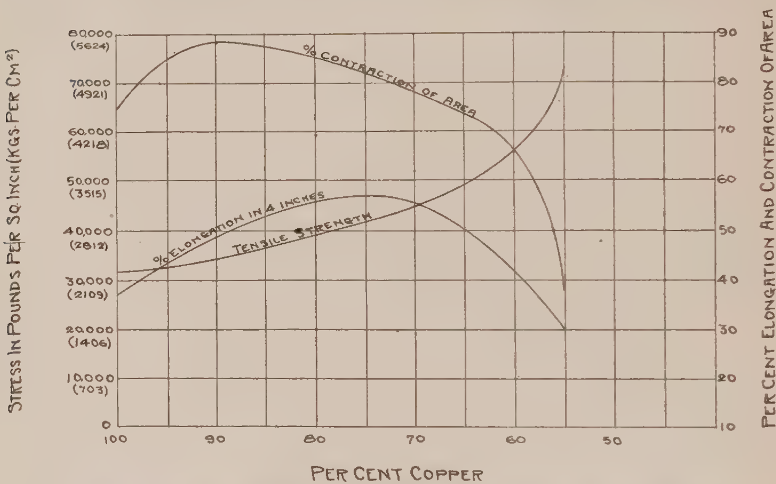

Fig. 3. Curves Showing Physical Properties of Cold Rolled and Annealed Copper-Zinc Alloys.

casting occurs, the extent of exposure to contamination from the atmosphere of the melting chamber, and the size, shape and material of the mould. Moreover, the mixtures richer in copper are more sensitive to such influences than those containing much zinc, which fact accounts for the much closer agreement of the two curves over the range from 63 per cent to 55 per cent copper than from pure copper to 63 per cent. All of these modifications may occur when copper and zinc of the greatest purity are employed. Additional modifications occur due to the normal impurities carried by these metals in the

commercial state. Beyond this, heat treatment and manipulation produce still further changes in physical properties.

Owing to the fact that important modifications of the properties of the brasses, as shown by Fig. 3, are obtained by the addition of other constituents (which, however, adversely affect ductility and malleability), they are not usually employed in the cast state without such additions. With wrought material, high ductility and malleability are of importance, while with cast material, they are not.

In connection with the curves shown in Fig. 3, it is necessary to consider the element of cost, which is influenced by

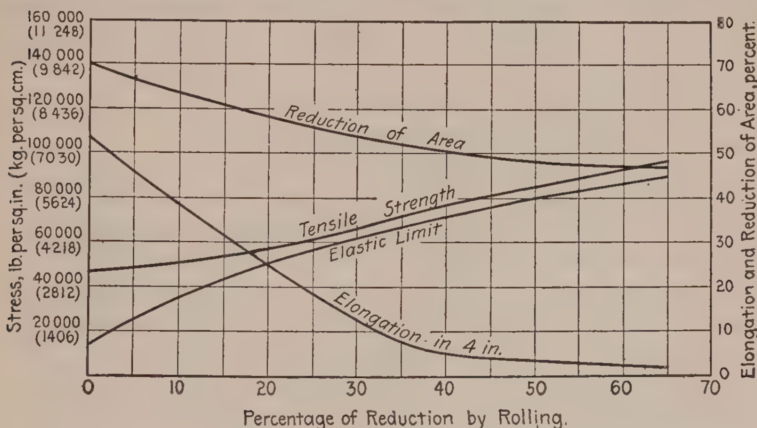

Fig. 4. Curves Showing Variation in Physical Properties of Rolled Sheet Brass, Due to Varying Amounts of Cold Rolling.

the fact that zinc is ordinarily obtainable at somewhat less than half the cost of copper. The entire range of alloys covered by the curves of Fig. 3 is capable of being usefully employed, either in the shape of castings made in sand moulds or in the wrought form. It is in the latter state, however, that the most extensive employment is found. The range from 63 per cent to 70 per cent copper covers those mixtures commonly termed, when in the wrought state, high brass. These combine in the highest degree the qualities of strength, uniformity, toughness and ability to withstand manipulation. They are not capable of being worked hot. They are most largely em-

ployed in the form of sheets and strips, seamless tubes and wire. To produce these forms, a blank of appropriate size and form is cast in an iron mould. This blank is then reduced to the finished size by cold rolling or by cold drawing. Both of these operations profoundly affect the physical properties. Fig. 4 shows the changes produced by various amounts of cold rolling upon a sample of brass containing 67 per cent copper and 33 per cent zinc (2). These curves were obtained by taking a sheet 0.10″ thick and carefully annealing it to about 550 deg. C. From it strips were cut and rolled to successively decreasing thicknesses. The strips were then submitted to tensile tests and the results coordinated with the degree of cold rolling, this latter value being expressed by $\frac{T - t}{T} \times 100$ where

T = initial thickness or 0.10″
t = final thickness after each rolling

The effect of cold drawing is similar to that of cold rolling.

Owing to the change produced by these operations, they cannot be continued indefinitely. It is, therefore necessary to anneal the material after the operation has been carried to a certain extent. This treatment consists of heating the material and allowing it to cool. The result of this treatment is to remove the effects of the cold working. Fig. 5 shows the effect of varying annealing temperatures (2). The element of time is not important, except over relatively long periods.

The value of the high brasses is due, in addition to their resistance to corrosion, to their ability to withstand forming operations, and to the very wide range of properties they may be made to possess by submitting them to varying degrees of annealing and to varying amounts of cold working. They may, as a final process, be annealed to low, medium or high temperatures, or cold rolled 20 per cent, 40 per cent, 60 per cent or more, according to the requirements of the use for which they are intended.

Mixtures between 63 and 55 per cent copper comprise what is known as Muntz metal and possess three distinctive qualities,—ability to work hot, relative cheapness on account of the high percentage of the relatively cheap zinc, and high

strength. The first of these qualities is taken advantage of in connection with the manufacture by hot processes of products the uses of which do not demand very high ductility. They have certain limitations. One of them is inability to work readily cold. Another is due to the greater difficulty of producing these products with uniform characteristics. Reference to Fig. 3 will show that the physical characteristics vary rapidly with the zinc content. This is difficult to control within narrow limits because zinc is volatile and the loss on

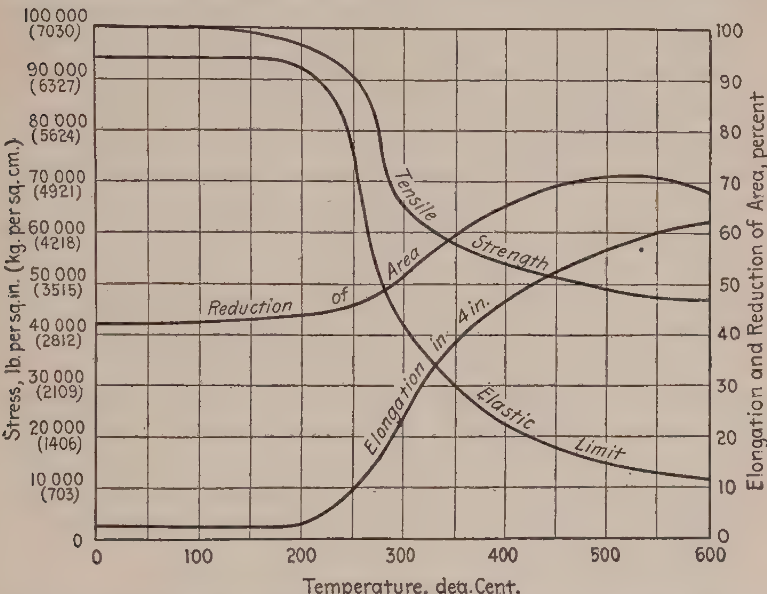

Fig. 5. Curves Showing Variation in Physical Properties Due to Annealing Hard-Rolled Brass at Varying Temperatures.

this account during the operation of melting is variable. This is the more important because brass is cast in small lots from crucibles, so that any large quantity will consist of many heats. Seamless tubes of Muntz metal are made by modifications of the Mannesmann process, by means of which solid cylindrical billets are pierced by hot rolling and subsequently finished by cold drawing. The extrusion process is also employed in the manufacture of this class of products to a limited extent. Rods and sheets of Muntz metal are also made by hot rolling.

The extrusion process was originally developed in connection with the production of rods of irregular cross-section, such as could not readily be made by rolling or drawing. It is in connection with the manufacture of leaded brass rods for use in automatic screw machines, however, that this process has had its largest employment. Lead, up to three or more per cent, is added to Muntz metal, forming a mixture which can be easily machined at very high cutting speeds and can, at the same time, be readily extruded. As this product is shaped by cutting tools and not by processes involving distortion, the low ductility characteristic of Muntz metal is not detrimental.

For purposes where high strength is essential, it is usual to employ Muntz metal alloyed with small percentages (up to 1½ per cent) of tin or of tin and iron. The former mixture is the basis of the rolled bronze of the U. S. Navy specifications and of various proprietary bronzes, while the latter constitutes manganese bronze, which usually contains a few per cent less zinc than rolled bronze as well as a small amount of manganese. Both of these materials may be produced by hot rolling and also by extrusion, but are usually finished to size by cold drawing. Their manufacture involves the exercise of a high degree of skill because of the difficulty of maintaining the composition within close limits and because of their sensitiveness to variations in heat treatment. Failure in these respects causes wide variations in physical properties, which may be expected to depart to a considerable extent from the highest attainable even under the best conditions.

As these materials are usually employed for members subjected to severe stress, a full knowledge of the loads which they may be expected to bear is important. The U. S. Navy specifications in the case of rolled bronze call for a minimum tensile strength of 60,000 to 62,000 lbs. per sq. in. (4200 to 4350 kg. per sq. cm.), a yield point of one half the tensile strength and an elongation of 25 to 28 per cent in 2″ (5.1 cm.); and in the case of manganese bronze, from 70,000 to 72,000 lbs. per sq. in. (4900 to 5050 kg. per sq. cm.) tensile strength, an elastic limit of one half the tensile strength and 28 to 30 per cent elongation in 2″ (5.1 cm.). The shape of the elastic curve of these materials, in common with that for most of the copper

alloys, is, however, such that accurate determinations of the elastic limit or yield point can only be obtained by the use of refined methods. It is, in fact, questionable if values as high

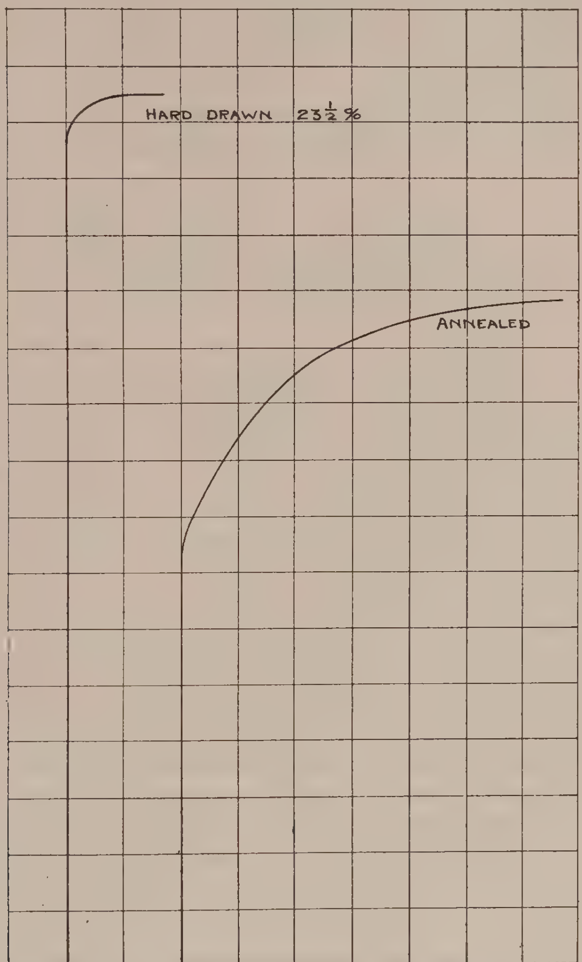

Fig. 6. Stress-Deformation Diagrams of Wrought Manganese Bronze.

as those called for above can be regularly met when highly accurate determinations are made, commercial methods usually placing the values at too high figures. Capp(3) has called

attention to this fact, although his experiments were apparently made on material not of the highest quality. Fig. 6 shows autographic stress-deformation diagrams of a high quality manganese bronze, one sample of which was tested in the annealed condition, the other, after having been reduced 23½ per cent by cold drawing. The properties of these two samples were as follows:

Condition	Tensile Strength	Elastic Limit	% Elongation in 2″ (5.1 cm.)
Annealed	76,700 lbs. per sq. in.	48,200 lbs. per sq. in.	33½
	5,380 kg. per sq. cm.	3,370 kg. per sq. cm.	33½
Cold drawn	99,400 lbs. per sq. in.	94,500 lbs. per sq. in.	9
	6,980 kg. per sq. cm.	6,650 kg. per sq. cm.	9

These tests demonstrate the way in which both the tensile strength and elastic limit can be raised by cold drawing or by cold rolling. This practice, however, produces internal strains in the material which may subsequently cause it to develop serious cracks, a phenomenon known as "age" or "season" cracking. This action has been investigated by Heyn(4) and by Flinn(5).

The mixtures containing more than 70 per cent copper are employed for special uses, such as those in which color or high ductility are essential considerations. Those mixtures containing about 80 per cent of copper are designated "low brass", those with 90 per cent of copper, "commercial bronze", and those with 95 per cent copper, "gilding".

The properties heretofore discussed are those normal to mixtures containing copper and zinc, with the least admixture of other elements. Important modifications are produced by the presence of other constituents, the chief of which is lead. This is carried as an impurity in the zinc, and also added intentionally. It has two important effects,—one is to reduce ductility, the other is to render the brass more readily acted upon by cutting tools. Zinc is commercially obtainable in three different grades, containing about 0.10 per cent, 0.20 per cent, and 0.60 per cent lead respectively; the latter grade being more generally employed; while the two former, which are more

expensive in inverse proportion to the lead content, are employed only where maximum toughness and ductility are demanded. In addition to the amounts covered by the zinc, lead is often added, as above mentioned, for machining purposes,—this property being acquired at the expense of toughness. One half of one per cent has an appreciable effect while with two per cent a marked difference in this respect is obtainable. Large amounts of lead seriously reduce the ductility of the material, this effect increasing with the copper content; so that mixtures rich in copper cannot readily be produced containing high percentages of lead.

Tin is sometimes added to wrought brass for the purpose of increasing its strength and elasticity and also to render it less susceptible to corrosion. Unless used to excess, it has no deleterious effect.

Iron is always present to some extent as an impurity, and increases strength and elasticity but seriously reduces ductility. When the latter property is important, it should not exceed 0.10 per cent.

Next to the brass alloys in importance, are the bronzes, which consist of copper alloyed with tin. The effect of the addition of tin to copper is somewhat parallel to that of zinc. The increase of strength with decrease of copper content takes place more rapidly, however, reaching a maximum with between 15 and 20 per cent tin. Fig. 7 shows the variations in physical properties of cast bronze as determined by Shepherd and Upton(6). The bronzes containing 10 per cent or less tin can be cold rolled and cold drawn, but cannot be worked hot when containing much over 2 per cent tin. Fig. 8 shows the properties of annealed wrought bronze as determined by the writer. These are not parallel to the values obtained from cast bronze, due to the fact that careful heat treatment and fluxing are necessary for the production of this material in wrought form. In fact, it is essential to the production of satisfactory bronze castings that some flux be employed. The material generally employed is phosphorus, forming phosphor bronze. This material has an extensive application in castings where hardness, strength and reliability are required. Even when skillfully made, however, phosphor bronze castings show wide

variations in strength. Philip(7) publishes the results of a large number of tests of castings of phosphor bronze ranging from 5 per cent to 15 per cent of tin content. These tests show wide fluctuations in tensile strength and elongation, even when the compositions agree closely with each other. He proposes, as provisional specifications for this material, for large castings a maximum of 90 to 92 per cent copper, tin 8 to 10 per cent, phosphorus 0.3 to 0.6 per cent, with a tensile strength not less than 38,000 lbs. per sq. in. (2670 kg. per sq. cm.), and an elongation of not less than 20 per cent in 2″ (5.1 cm.). The per-

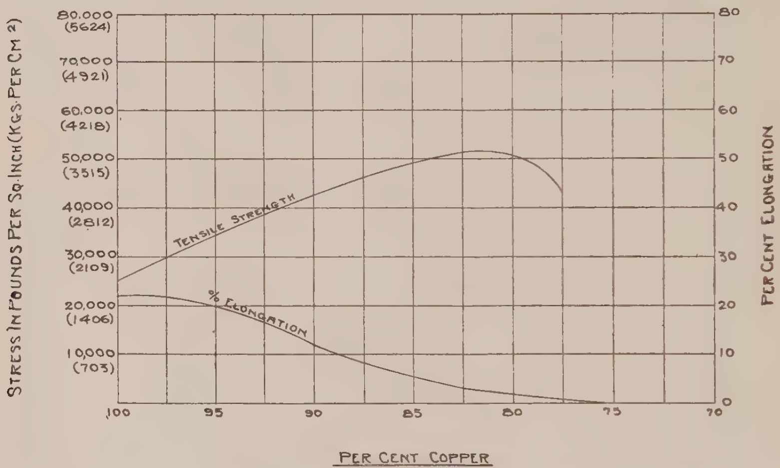

Fig. 7. Curves Showing Physical Properties of Cast Copper-Tin Alloys as Determined by Shepherd and Upton.

centage of phosphorus which it is desirable for a casting to contain is not well established. Its function is that of a deoxidizer, and for this purpose it is necessary that only a very small quantity be present in the casting. Some authorities, however, consider that the best results are obtained when it is present in amounts up to 0.70 per cent.

On account of the cost of the mixture as compared with brass and the expense of manufacture, phosphor bronze in wrought shape is not extensively employed. In the form of hard rolled sheet and hard drawn wire, however, it is valuable for making springs, being much superior for this purpose to any brass mixture. Moreover, it is not susceptible to age cracking.

There are many uses for which castings containing copper, zinc, lead and tin are employed; strictly speaking, these castings are neither bronze nor brass. These are usually known as red brass. By varying the proportions of the constituents, a wide range of strength, toughness, color, cost and machining qualities may be obtained. The range of proportions most usual is 76 to 87 per cent copper, 3 to 6 per cent lead, 2 to 7

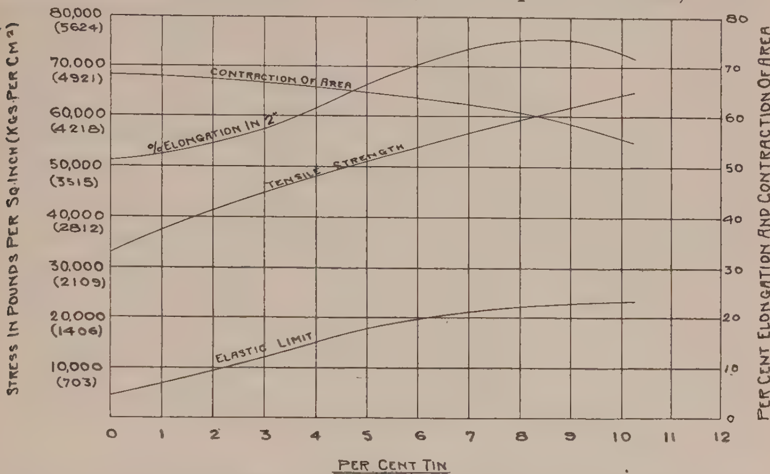

Fig. 8. Curves Showing Physical Properties of Cold Rolled and Annealed Copper-Tin Alloys.

per cent tin, and 3 to 16 per cent zinc. When low cost is desired, it is usual to decrease the copper and omit the tin, such mixtures being known as yellow brass. It is not possible to predict with accuracy the physical properties of castings, so that large factors of safety are necessary in connection with them. This fact is well shown by the investigations of Karr([8]). These were undertaken for the purpose of establishing a standard test bar for use in connection with castings of gun metal containing 88 per cent copper, 10 per cent tin, and 2 per cent zinc. This is a standard mixture of great utility, yet his tests show individual tensile strengths from 29,300 lbs. per sq. in. (2050 kg. per sq. cm.) with 9.5 per cent elongation in 2″ (5.1 cm.) and 16.9 per cent contraction of area, up to 50,500 lbs. per sq. in. (3550 kg. per sq. cm.), 53 per cent elongation in 2″ (5.1 cm.) and 40.3 per cent contraction of area. A number of

elastic limit determinations made in connection with this investigation show values from 15,000 to 17,000 lbs. per sq. in. (1050 to 1190 kg. per sq. cm.) with one sample showing the extreme figure of 20,000 lbs. per sq. in. (1400 kg. per sq. cm.). Good specifications for this mixture usually call for a tensile strength of 30,000 lbs. per sq. in. (2100 kg. per sq. cm.), a yield point of 15,000 lbs. per sq. in. (1050 kg. per sq. cm.), and an elongation of 15 per cent in 2″ (5.1 cm.), but it will be readily understood that in complicated castings portions may exist where lower values may be expected. Mixtures containing copper, tin and lead are extensively employed for machinery bearings. In cases where heavy pressures are encountered a proportion of 80 per cent copper, 10 per cent tin and 10 per cent lead, fluxed with phosphorus, is standard. In cases where conditions do not call for ability to resist shock, superior results are obtained by greatly increasing the lead and decreasing the tin and copper. A well known proprietary mixture successfully employs lead up to 30 per cent, but ordinary foundry methods will not permit the use of such percentages.

Manganese bronze forms a very useful material for castings requiring great strength and ability to withstand severe shocks. The mixture employed for this purpose is the same as that used for this product in the wrought form, except that a fraction of a percent of aluminum is added. Approved specifications call for a tensile strength of 65,000 lbs. per sq. in. (4,550 kg. per sq. cm.), an elastic limit of 30,000 lbs. per sq. in. (2100 kg. per sq. cm.), and an elongation of 15 per cent in 2″ (5.1 cm.).

Copper is employed in connection with aluminum to form aluminum bronze, in proportions up to 10 per cent of the latter. The strength increases with the latter element up to a high figure, although those proportions which give the highest strength, upwards of 85,000 lbs. per sq. in. (6000 kg. per sq. cm.), are lacking in ductility. Lesser proportions of aluminum give lesser strength, but greatly increased ductility. All percentages can be cast and worked both hot and cold, but require the exercise of great skill to yield satisfactory results. As a consequence, they have not met with extensive employment.

Nickel and copper readily alloy and yield products having useful qualities, but are expensive on account of the high cost of nickel. Compositions containing from 15 to 20 per cent of nickel are known as cupro-nickel and meet with somewhat restricted uses in the wrought form. They resist corrosive agencies well, and have a silvery color. A special proprietary alloy known as Monel metal is very useful in both wrought and cast form because of its great strength and ability to resist corrosion. In the latter state it has a tensile strength of upwards of 65,000 lbs. per sq. in. (4550 kg. per sq. cm.), an elastic limit of 32,000 lbs. per sq. in. (2250 kg. per sq. cm.), and an elongation of 25 per cent in 2″ (5.1 cm.). In wrought form it will give figures about 30 per cent higher than the above. It is a natural alloy, in that the copper and nickel are mixed in the ore from which it is smelted. Its composition is approximately 68 per cent nickel, 2 per cent iron, and 30 per cent copper. It has a very high melting point and is accordingly somewhat difficult to work.

The writer desires to acknowledge his indebtedness to Mr. G. H. Clamer and Professor Lionel S. Marks for their assistance and advice in connection with the preparation of this paper.

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Paper No. 112

THE ENGINEERING USES OF ALUMINIUM.

By

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It is less than a century since aluminium was first isolated from its compounds, a little over half a century since it ceased to be a chemical curiosity and began to be used, and barely a quarter of a century since it began to be counted as an ordinary metal of every-day life.

To be more specific, in the first of the above periods, 1827 to 1854, it cost more than its weight in gold to produce it, and it was put to no uses whatever; in the second period, 1854 to 1889, its selling price was $10 to $5 per pound, and it was used mostly for objects of luxury and in places where cost was a very secondary consideration; in the third period, 1889 to 1914, its price has dropped from $2.50 down to $0.20 per pound, and it has found large and important applications in the most varied lines.

The engineering uses of aluminium are based on its valuable physical and chemical properties, and on its being available to industry in large quantities at a moderate and practicable price. In January, 1915, it was quoted in New York at 18 cents per pound, and in Europe at 15 cents (1.65 francs per kilogram). As to the supply: before war began in 1914 the world's production was at the rate of 50,000 tons per annum, with prospects of 75,000 tons production in 1915 and 100,000 tons in 1916. The producing capacity of the aluminium works now in operation and projected will assure an abundant supply of the metal for several years to come.

Since all the uses of aluminium are based on particular physical or chemical properties, or collection of properties, it will be most practical to enumerate or describe these proper-

ties, and under each one to discuss the particular uses based on each property. Since the properties of aluminium can be extensively modified by alloying it with small proportions of other metals, we will make a third division of the subject, under which to discuss the alloys and their uses. It must be borne in mind that commercially pure aluminium, unalloyed, is never absolutely pure aluminium, but contains usually ½ to 1 percent of impurities, the principal of which are iron, silicon and copper. In what is called No. 1 aluminium these impurities do not exceed 1 percent; in No. 2 metal they may reach 3 percent. The purer the metal the softer and weaker it is, and, in general, the less subject to corrosive influences.

The contents of this paper will therefore comprise:

I. Physical properties of aluminium and uses based thereon.

II. Chemical properties of aluminium and uses based thereon.

III. Aluminium alloys, and their particular uses.

The question as to what constitutes an "Engineering" use is not easy to define, and may lead to some misunderstanding. Let us define it as any use which employs it as part of a structure, machine or apparatus or for a purpose useful to the engineering profession.

PHYSICAL PROPERTIES OF ALUMINIUM AND USES BASED THEREON.

1. Specific Gravity.

The lightness of aluminium has been its chief claim to attention for half a century. No. 1 commercial aluminium has the following density at 17°C = 62° F.

Cast	2.56	
Drawn or rolled, unannealed	2.68	
Drawn or rolled, annealed	2.66	
Weight of 1 cubic inch, cast	0.092	lb.
" " " " rolled, annealed	0.097	"
" " 1 cubic foot, cast	159.63	"
" " " " rolled, annealed	167.11	"

Compared with other common metals, this low density is remarkable in a metal which otherwise has many of the me-

chanical properties of copper or brass. A comparative statement of this matter follows:

	Density (Water = 1)	Relative Densities
Aluminium	2.68	1.00
Zinc	7.19	2.68
Tin	7.29	2.72
Wrought Iron	7.70	2.87
Rolled High Brass	8.55	3.19
Nickel	8.80	3.28
Rolled Copper	8.93	3.33
Silver	10.53	3.93
Lead	11.37	4.24
Gold	19.32	7.21
Platinum	21.50	8.02

The very practical consequence from this low specific gravity is that wherever lightness is a primary consideration, aluminium replaces many other metals to great advantage. And, since cost must always be regarded, it is important to remark that in such replacement it is not the cost of equal weights (per pound or per kilogram) which controls, but the cost of equal volumes. One pound of aluminium sheet will have 3.19 times the area of one pound of brass sheet of the same thickness, and therefore the price of $\frac{1}{3}.19 = 0.313$ lb. of aluminium sheet must be compared with the price of 1 lb. of sheet brass, to determine their relative costs for the same quantity (volume) of sheet metal.

Even when aluminium sold for $10 per pound (1859 to 1885) it was used for many articles of luxury where this exceptional lightness was a great advantage, such as in opera glasses, field glasses, compasses, sextants, helmets, flasks, cameras, cigar cases, match boxes, and numerous other kinds of portable objects.

To the engineer who has much traveling to do, carrying instruments and equipment, the substitution of aluminium for other metals, wherever possible, is strongly to be recommended, as it enables saving one-half to two-thirds of the weight of metal carried.

2. Color.

Aluminium is white with a very faint bluish tinge. When frosted or roughened, as by dipping a moment into caustic

alkali solutions or nitric acid, or by use of the sandblast, it becomes satiny white. When beaten out into very thin foil it replaces silver foil for silvering, and when this is ground to fine powder it makes an excellent metallic paint, which is in general use for plumbing, radiators, letter boxes, exposed iron-work of every kind. This paint should **not** be used for radiators, because its color diminishes considerably the amount of heat which the radiating surface can disperse into a room; but for this very quality it is an excellent paint for steam boilers, furnaces, refrigerators and wherever it is desired to prevent radiation of heat. Knowing that a white metallic surface radiates very little heat, and also absorbs very little heat from outside, engineers should be scientifically discriminating in the uses of white aluminium paint. Where the white color of aluminium is objectionable in a structure, it may be painted any desired color. Paint does not adhere well to smooth, polished aluminium, but sticks excellently if the surface is first roughened or frosted, as by dipping in nitric acid or by the sandblast. (In fact, frosted aluminium takes paint or color so well that plates of aluminium form very good printing surfaces in place of lithographic limestone.) This is a very useful property where aluminium is used in automobiles, and similar uses, where it is desired to paint it the same color as the rest of the structure.

3. Conductivity for Heat.

Among the common metals, only copper is a better heat conductor than aluminium. Relatively, if copper is called 100, aluminium is about 50 and iron 15. This property of copper is of importance where liquids are to be heated or particularly when they are to be boiled, as in cooking, preserving foods, evaporating. For such purposes copper has for centuries been the metal most employed, but aluminium can now be used in place of copper at less cost. Although not so efficient a heat conductor, yet it possesses the great practical advantage of being non-poisonous. Many fatal accidents have occurred from poisoning by copper salts formed by food cooked in copper utensils, while aluminium is either not attacked by the food or the salts formed are harmless. This great advantage over copper more than offsets its lower heat conductivity, which is still

high compared to that of other common metals. It results, that aluminium cooking utensils have come into common use, in enormous quantities, being not only cheaper than copper but perfectly safe to use, and in addition, only one-third the weight to handle, when empty.

Besides ordinary household and kitchen use, which may not be dignified by the term "engineering," the chemist and the food packer find large use for aluminium in steam-jacketed vessels of the most varied kinds, for boiling, digesting, evaporating and heating all kinds of liquids, sauces, condiments, fruits and foods; also in vacuum pans, stills and condensers, where its high heat conductivity is of practical importance.

4. Expansion by Heat.

Between 0° and 100°C. (32° to 212°F.) aluminium expands 0.0000231 of its length for each 1°C., and 0.0000130 of its length for each 1°F., its length at 0°C. being taken as unity. This is nearly double the expansion of iron, and half again as large as copper.

From the liquid state, when cast, pure aluminium contracts 0.017 of its length (17 mm. per meter, 0.203 inch per foot), in castings about 6 mm. (0.25 inch) thick. Thicker castings will shrink slightly less, and thinner castings more.

5. Weldability.

On heating to low redness, just below its melting point, aluminium becomes granular and brittle (short). It cannot be welded in the ordinary sense, like iron, by hammering below its melting point. But it is quite capable of being welded by the process known as autogenous welding. This is similar to the old art of "lead-burning", where two metallic edges are placed together, or slightly overlapped, and the joint is made by melting the edges together by a blowpipe flame. If the sheets are heavy, a rod or strip or wire of pure aluminium is laid in the joint or is used like a bar of solder and melted into the joint. Flux must be used: that of Schoop consists of dry potassium fluoride and potassium bi-sulphate; others use a mixture of alkali fluorides. The flame is sometimes oxygen-hydrogen, or oxygen-coal gas, or oxygen-acetylene, or air-acetylene. Workmen soon become expert in this work, and

seams of any length can be made on material up to 1 cm. (0.4 inch) in thickness, and down to the thinness of paper. Large vessels or tanks can thus be built up in practically one piece of metal, with absolutely no danger of corrosion or leaking at joints. This method has largely displaced riveting, and almost altogether displaced soldering.

By making the blowpipe apparatus portable, welding can be done anywhere, and aluminium cables can be spliced or bus-bars joined in place, into one continuous piece, in any length desired. Continuous metallic belts or bands or carriers can be thus made, or aluminium wire or sheet furnished in continuous lengths to any length desired.

6. Extrudability.

The softness of aluminium, and its workability, give it the valuable property of extrudability, by virtue of which it can be pressed while warm through a die, by great pressure, and extruded into elongated shapes of any desired cross-section, however complicated, and to practically any length desired. The apparatus used is similar to that used for extruding lead pipe, but stronger; machines at present in use produce extruded shapes up to 6 inches (15 cm.) in diameter, with walls as thin as ⅛ inch (3 mm.). Continuous tubing of any length can be also thus made, or continuous bus-bars of great length. Very large quantities of fancy section angles, rods and mouldings, for automobiles, aeroplanes, building and architectural work, machinery, etc., are now produced by extrusion, and sold at but a small advance on the price of ingot aluminium, thus greatly extending the uses of this metal.

Engineers will certainly make large use of these extruded shapes, which practically take the place of rolled shapes of all cross-sections, except when over 6 inches in cross-section. For the larger cross-sections, rolling must at present be used, but since the extruded metal is mechanically superior to rolled metal, it is to be hoped that extruded shapes of large size may be soon available to engineers. The variety of cross-section shapes at present available is surprisingly large (over 250), and it may be said that any desired shape of cross-section may be had by the consumer who is willing to pay for the necessary dies, which cost $25 to $300 per set.

Engineers will be interested in knowing also that some of the stiff aluminium alloys can also be extruded, giving some shapes of great mechanical strength, and that some alloys can be extruded which it is not possible to roll.

7. Conductivity for Electricity.

The electrical engineer is particularly concerned with the large conducting power of aluminium. Here again copper is the only common metal with higher conducting power, and it is copper which aluminium is rapidly replacing. The exact figures for hard-drawn copper and hard-drawn aluminium are 97 and 61 respectively; pure copper, Matthiessen standard, being 100. As a first approximation one may say that to replace a copper wire by one of aluminium of the same conducting power, increase the diameter one-fourth. This increases the cross-section 56 percent, and gives practically equal carrying power.

$$\left(\frac{97-61}{61} \times 100 = 59 \text{ percent.}\right)$$

The weight of the aluminium conductor will be $159 \div 3.33 = 0.48 = 48$ percent of the weight of the copper it replaces. As a practical basis for this replacement we may therefore figure on 60 percent increased cross-section, and one-half the weight. Since aluminium is now sold regularly at less than double the price of copper, we may say that at present aluminium is a cheaper electrical conductor than copper.

Aside from the mere cost of the metal in the conductor, there are a number of factors which influence the question of the substitution of aluminium conductors for copper. They are mostly practical engineering considerations. Some of them are discussed at length in a paper by H. W. Buck (Proceedings International Electrical Congress, St. Louis, 1904) and in a monograph by E. Dusaugey (Les Conducteurs d'Electricité en Aluminium, Paris, 1912); the Aluminium Company of America has also issued a 50-page pamphlet on this subject. When we mention that the monograph of Dusaugey has 140 pages, it will be seen that any detailed discussion of the advantages, disadvantages and installation of aluminium conductors is far outside the scope of this paper. We will, however, mention the main items under consideration.

A. Where space is restricted, as in the windings of dynamos and motors, the larger section of aluminium is sometimes inadmissible. This is to some extent overcome by the method of insulating aluminium with a superficial thin chemical compound, and dispensing with the ordinary thick insulation.

B. Bus-bars in chemical plants often give trouble at connections. Aluminium is harder to keep clean and has lower resistance at the joints than copper. Copper can be satisfactorily tinned and soldered at such joints; aluminium cannot. If the aluminium joint is autogenously soldered, such difficulties disappear; if not, it must be bolted more tightly and more frequently cleaned, than copper.

C. Aerial wires for power transmission can be satisfactorily joined by the twist joint. The two ends are slipped into a flattened aluminium tube, and with powerful pliers are given a half-dozen twists. Such a joint has and retains greater conducting power than the wire itself. With heavy cables a different method is used. The two ends are inserted into a cast aluminium sleeve of proper size, and the sleeve then subjected to great pressure in a die in a hydraulic jack. The metal of the cable and the sleeve thus flow together into a solid mass. The jack can be made portable, and such joints made out in the field; the whole apparatus weighs only 200 pounds. Sometimes the compression is applied in the factory, a screw socket being thus welded on the end of a cable, enabling it to be attached to a similar screw socket by a double-threaded stud, whenever connection is to be made.

D. The cost of insulation on an aluminium wire will be some twenty-five percent greater than on copper. But large cables for power transmission are usually strung without insulating covering; in which case this factor does not enter.

E. Hard-drawn aluminium wire being about half as strong as hard-drawn copper wire, per unit of section, the aluminium conductor with 60 percent greater cross-section than copper is really 0.8 as strong as the copper wire which it replaces. This is, however, more than offset, in practical use, by the fact that the aluminium is only half the weight of the copper. For spans of equal length the weight of aluminium to be carried is only 50 percent that of the copper, while its strength is 80 percent;

it is therefore relatively 1.6 times as strong. Or, putting it another way, the length of span between supports could be increased approximately 50 percent, with the same dead-weight on the supports and the same factor of safety on the strength of the wire.

F. The increased span decreases the cost of the supports or towers on a line, the number of insulators, and the cost of erection; also the leakage to the ground through the insulators and towers. These savings are in general far more important than the saving in cost of the cable itself. In fact, the aluminium cables might be profitably used even if the wire itself cost more than copper.

G. A combination cable consisting of aluminium wires twisted upon a steel core has even greater advantages than the plain aluminium cable. Taking a hard-drawn steel wire of 160,000 lb. per sq. inch ultimate tensile strength and 130,000 lb. elastic limit, and twisting tightly upon it six aluminium wires of equal diameter, the hard-drawn aluminium having 24,000 lb. tensile strength and 13,000 lb. elastic limit, there is obtained a combination which has only 80 percent of the weight of a copper cable of equal conductivity but which is 57 percent stronger. With about equal cost for cable it is possible to make so much longer spans with this composite cable that there are great savings in cost of erection, towers, insulators and leakage. Almost all the new high-power transmission lines are being constructed with this composite cable. The steel wire is galvanized before use, and the aluminium covering is twisted upon it so tightly as to practically protect it from air and moisture.

H. The calculation of wind stresses shows that more allowance must be made for these than for the smaller copper cable.

I. Corrosion of the aluminium is found to be no greater than that of copper cable under the same conditions. Near the sea-coast, both aluminium and copper are strongly corroded by the salt moisture and they should both be insulated.

J. Where the line is at a great distance from the manufactory, the saving in freight on cable and insulators may be an important item.

K. Scrap aluminium is much less easy to melt up or to dispose of surreptitiously than scrap copper. It is therefore much less likely to be stolen—a fate which sometimes befalls copper cable.

8. Mechanical Strength.

The commercially pure metal is not strong. It is a soft metal with low tensile strength and elastic limit. For No. 1 metal, 99 percent pure or over, the following are average figures:

	Cast Metal	Sheet	Rod	Wire
Elastic limit, in tension,				
lb. per sq. in.	8,500	12,500	14,000	16,000
		25,000	23,000	33,000
kg. per sq. mm.	6.0	8.8	10.0	11.6
		18.1	16.7	23.9
Ultimate strength, in tension,				
lb. per sq. in.	12,000	24,000	28,000	25,000
	14,000	40,000	40,000	55,000
kg. per sq. mm.	8.7	17.4	20.3	18.1
	9.9	29.0	29.0	39.6
Reduction of area, percent	15	20	30	40
		30	40	60

By working, aluminium becomes hard and stiff, like hard-drawn copper or brass. It has a large range between elastic limit and ultimate strength, which leaves large margin in which it can be worked, i. e., its shape permanently changed without rupturing the metal.

In compression, cast aluminium has the following properties:

	lb.	kg.
Elastic limit, in columns, length twice diameter	3,500	2.5
Ultimate strength	12,000	8.7
Modulus of elasticity	9,000,000	6,500

Where strength is a desideratum, and the metal must be stiffer or stronger, alloys are used containing small amounts of other metals. These are very interesting bodies whose mechanical properties will be given under a separate heading.

It should here be brought to the attention of the engineer, however, that if the aluminium is substituted for a strong metal in a construction, as in a beam or girder, the aluminium gains

because of its greater volume. If, for instance, an aluminium girder is substituted for steel, with exactly the same cross-section, the aluminium is much weaker, but if it is made of the same weight, it will be much stronger, because it can be given three times the cross-sectional area. Taking standard section of I beams, for example, an 8-inch steel beam will weigh as much per foot as a 15-inch aluminium beam, but the latter will carry 80 percent more working load. Where the space can be taken, therefore, the aluminium can be very satisfactorily used; or conversely, the same strength can be obtained by using greater cross-section but less weight, and thus weight saved at the expense of using up more space. This is often practicable in the bracing of structures internally, where the space occupied by the bracing is of no importance.

CHEMICAL PROPERTIES AND USES BASED THEREON.

Action of the Weather.

Moist air rusts aluminium, not as quickly as it does iron or steel, but about as quickly as it does copper. An apparent exception to this statement is the white aluminium paint, which is weather-proof, but this is because the aluminium in it is ground up in presence of grease, which coats each particle and protects it from oxidation.

The oxidation of polished aluminium takes the form of a thin white coating, quite continuous and adherent. It changes the color of the metal to a gray, and sometimes shows up spots like cigar-ashes where the metal is not uniform in composition. Greasing or lacquering of the polished surface protects it well from oxidation, just as it would polished iron or copper. It can be re-polished by using bath-brick or sapolio, finishing off with silver polish (tripoli or electron) if a high polish is desired.

Paint does not adhere well to polished aluminium, and if it is desired to protect with paint the surface should first be roughened or frosted, either by an acid wash or by sand-blasting. Oil paints adhere best. Aluminium surface can also be chemically blackened, by a copper sulphide coating (gun-metal or black-iron effect), giving a surface quite resistant to the weather or to the effects of ordinary handling. This is frequently applied to sextants and surveyors' instruments.

Action of Chemicals and Foods.

Only hydrochloric acid and caustic alkali solutions act strongly upon aluminium. Other strong acids will act on it strongly in the presence of salt. Soda solutions attack it and blacken it; they should be kept away from aluminium vessels. The highly seasoned curries of India are said to be hard on aluminium cooking utensils. The most severe combination it is likely to meet in ordinary cooking is strong vinegar and salt. Strongly acid tomato stew has some action on it, mostly in the direction of cleaning, for the utensil is brightened up inside by cooking tomatoes in it. On the whole, however, all common foods can be said to be without perceptible chemical action on aluminium, so that such cooking utensils are practically indestructible. This is a valuable property in the equipment of a mining camp, a surveying party or an exploring expedition. In recent years, exploration parties, particularly arctic and antarctic, have been equipped with aluminium cooking utensils, also all modern armies, including not only the regular army kitchens but the cup, plate and bowl of the individual soldier. If these are not, strictly speaking, engineering uses, yet they are uses which are very valuable to the various kinds of engineers.

LIGHT AND STRONG ALUMINIUM ALLOYS.

For half a century many and various attempts have been made to alloy aluminium in such a way as to greatly increase its strength without greatly increasing its specific gravity, so as to give to the engineer a strong, light construction material.

A considerable number of metals harden and strengthen aluminium, without increasing much its specific gravity. In fact, it is a useful fact, that the heavier metal added increases the specific gravity not in proportion to the weight of alloying metal used but in proportion to its volume. Ten percent by weight of zinc, for instance, does not raise the specific gravity one-tenth of the way from that of aluminium to that of zinc, but one twenty-seventh of the way (27 to 10 being the relative specific gravities of aluminium and zinc). As early as 1860 scale makers stiffened aluminium with silver, in order to make

rigid balance beams, and since then a large number of alloys satisfactory for various purposes have been found.

Alloys with Copper.

		Tensile Strength lb. per sq. in.	Elastic Limit lb. per sq. in.	Elongation %
2 to 3 percent copper casts very well; works well.				
6 percent rolls well.	Rolled hard	35,000		3.5
	Annealed	25,000		15.5
	Casting	21,500	12,000	1.5
2 percent Copper, 0.5 " Nickel, 0.5 " Zinc	Cast	22,000		
	Rolled	40,000		4.0

Alloys with Zinc.

		Tensile Strength lb. per sq. in.	Elastic Limit lb. per sq. in.	Elongation %
10 percent Zinc	Cast	19,850	4,000	1.5
15 " "	Cast	21,780	14,000	
20 " "	Cast	24,140	16,000	0.4
33 " "	Cast	34,060	24,000	0.4
	Rolled	53,000		4.0
15 percent Zinc, 3 " Copper, 0.5 " Manganese	Cast	29,415	16,000	0.5
30 percent Zinc, 1.5 " Copper	Cast	42,500	25,000	0.8

It may be remarked of these zinc alloys that they are particularly good for castings, and that the cheapness of zinc and its large amount makes them cheaper than any other light aluminium alloys. They machine freely, like brass, and resemble bronze in their rigidity. Their specific gravity is 2.84, 2.93, 3.02 and 3.31 for the 10, 15, 20 and 33 percent alloys respectively. Their melting points are all slightly below that of aluminium.

These are the alloys, particularly those with a little copper, which closes the grain and makes a finer texture, which are used mostly for motor-casing castings for automobiles. In fact, the making of these zinc alloy castings for various machine purposes, such as parts of gasoline motors and motor castings, has become a large industry, and consumes a large proportion, probably one-third, of all the aluminium made.

Alloys With Magnesium.

These are noted because of the specific property of being lighter than aluminium itself. Magnesium has a density of

1.72, against aluminium 2.58, and the alloys are therefore lighter than aluminium. The action of magnesium mechanically is somewhat similar to zinc, up to a limiting addition of 10 percent magnesium. The alloys are ductile, and increase in strength by cold-working. At present magnesium costs more than aluminium, and these alloys are therefore more costly than aluminium itself, and considerably more costly than the zinc alloys. Their specific gravity runs from 2.4 to 2.6 cast, 2.5 to 2.7 worked. They are known commercially as Magnalium.

	Tensile Strength lb. per sq. in.	Elastic Limit lb. per sq. in.
2 percent Magnesium	15,440	8,700
5 " "	17,850	13,090
10 " "	19,680	14,600

Duralumin.

Recently, an alloy which has remarkable mechanical properties has been sold commercially under the above trade name. It is said to contain 3.5 to 5.5 percent copper, 0.5 to 0.8 manganese and 0.5 percent of magnesium. Its specific gravity is the same as that of aluminium. As manufactured, however, it is not particularly strong, but it is given its best mechanical properties by heat treatment, consisting in heating to within 50°C. of its melting point and suddenly cooling, as by quenching, to 300° C. On being laid aside after such treatment the alloy is said to increase in strength, and to attain its maximum strength in about a week. Whether this information is all reliable the writer cannot vouch for, but he can verify the statement that metal called Duralumin and having the density of aluminium has shown 59,000 lb. per sq. inch tensile strength, with 8 percent elongation, as tested by the German Testing Bureau at Charlottenburg.

USES OF ALUMINIUM.

Aeroplane Parts
Aluminotype
Ammonia Condensers
Ankle Stiffeners
Artificial Limbs, Noses, etc.
Automobile Parts, as follows:

Castings.

Bearing Caps
Bonnet Strips
Clutch Cones and Covers
Cowls
Crank Cases
Differential Housings
Fan Blades, Brackets and Wheels
Floor Boards
Foot Boards
Gear Box Covers
Hand Hole Plates
Hood Ledges
Hub Caps
Instrument Boards
Intake Manifolds
Oil Pans
Pistons
Radiator Frames and Tanks
Running Boards
Starting Box Brackets
Steering Gear Cases and Covers
Steering Wheel Spiders
Switch Boxes and Covers
Tire Holders
Transmission Cases and Covers

Sheet.

Body Panels
Deck Plates
Tenders
Foot-rail Casings
Hoods
Hub Caps
Running Board Pads and Mouldings
Seat Compartment Covers
Scuff Plates
Tops

Extruded Sections.

Angle Moulding
Door Moulding
Drip Moulding
Tender Skirt Moulding
Garnish Moulding
Hood Ledge Moulding
Panel Moulding

Backs of Brushes and Combs
Baskets
Battery Plates
Beams for Balances
Beer-brewing Apparatus
Billiard Cues
Bits
Blades for Fans
Boot Trees
Bowls
Boxes for Collars, Cuffs, Powder, Ointment, Pills, etc.
Bracelets
Braces
Brackets
Bread Pans
Bridles
Brooches
Bullets for Mob Suppression
Business Cards
Buttons
Cab Bodies
Cake Pans
Camera Parts
Candle Moulds
Candlesticks
Candy-making Vessels
Canoes
Cans for Food
Canteens
Cards
Car-roofs
Cash Conveyors
Cash Registers
Caul Plates
Chafing Dishes
Chains
Cigar and Cigarette Holders
Cigar Moulds
Clasps on Rubber Garments
Coffee Pots
Coins
Colanders
Collar Boxes

Combs
Compass Cases
Condensers
Cooking Utensils
Cornets
Cups
Cuspidors
Dental Plates
Dog Chains
Drop Forgings
Drum Frames
Electrical Conductors
Elevators
Explosives
Fan Blades
Field Glasses
Flash Light Powder
Foil for wrapping Food
Food Containers
Forks
Friction Gearing
Fruit Cans
Fruit Evaporating Shelves
Galvanizing
Gas Stoves
Gear Casings
Glue Nets and Pans
Grape Juice Apparatus
Griddles
Hair Pins
Hat Pins
Harness Trimmings
Helmets
Hobnails
Horns
Horseshoes
Hose Connections
Insoles for Shoes
Kettles
Key Chains and Keys
Knife Handles
Lasts
Leaf
Levels
Lightning-arresters
Lithographic Plates
Match Cases
Medals
Megaphones
Milk Pans and Pitchers
Mine Cages and Skips
Mirror Frames
Models
Mouldings
Moulds
Musical Instruments
Nails
Napkin Rings
Oil Stoves
Ointment Boxes
Opera Glasses
Ovens
Padlocks
Paint
Paper Cutters
Passementerie
Patterns
Pen and Pencil Holders
Perfume Bottles
Picture Frames
Pigeon Tags
Pill Boxes
Pipes
Pitchers
Plaques
Plates
Pocket Rules, Scales and Levels
Pontoons
Powder Plates
Puff Boxes
Racing Shells and Sulkies
Railway Car Parts
Rectifiers for electric current
Reflectors
Refrigerator Linings
Rings
Rivets
Roasting Pans
Roller Skates
Rules
Sauce Pans
Scales
Scientific Instruments
Semaphores

Sextants
Shades
Shelves for Fruit Dryers
Shoe Horns, Lasts and Soles
Skates, Sleds, Sleighs
Soup Ladles and Kettles
Spectacle Cases, Chains and Frames
Spindles
Spoons
Spurs
Stearic Acid Pans, Pipes, etc.
Steel Castings
Stoves and Stove Pipe
Stirrups
Strainers
Submarines
Sugar Mill Apparatus
Surgical Instruments
Suture Wire
Tea Pots and Balls
Teething Plates
Telephone Switchboard and Apparatus
Telephone Transmitter Diaphragms
Telescopes
Toilet Articles
Tracheotomy Tubes
Transmission Lines
Traveling Cases
Trays
Trunks
Trusses
Tubes
Typewriters
Umbrella Parts and Holders
Vacuum Cleaner Parts
Visiting Cards
Waffle Irons
Walking Sticks
Watches and Watch Chains
Water Coolers and Heaters
Wattmeter Discs
Wireless Apparatus
Yacht Construction
Zinc Castings

DISCUSSION

Prof. J. W. Richards, in reply to various questions, said that at present cost of producing aluminium is 10 to 15c. per lb., and it is sold at 18 to 20c. per lb. There have been large decreases in the cost of production in the past twenty years, as much as $5 per lb. in a single jump, but any future decrease will be a matter of a few tenths of a cent. Prof. Richards.

If pure alumina could be obtained, greater cheapness would follow.

There is great difficulty in the reduction of aluminium because of its great affinity for oxygen; therefore, the ore must be first purified to aluminium oxide, and then this is reduced to the pure metal.

In regard to increased uses of aluminium, they are becoming greater inversely as the cost is becoming less.

Aluminium is polished very much like ordinary tinware with bath brick, plus elbow grease. Any common polish will serve the purpose.

The best alloys with magnesium seem to be those which have a low percentage of magnesium. Hence the best alloys with this metal are but little more expensive than aluminium. Possibly the price of magnesium may be reduced to $1 per lb. or even 50c. per lb., if the demand for aluminium-magnesium alloys becomes greater.

Rusting of aluminium is referred to on page 568; it corrodes about the same as copper. It can be painted, but it is best protected by a heavy coat of lacquer.

Aluminium is a very serviceable metal for certain uses, to which it is well suited. More uses for it are being discovered every day, and the development of the aluminium industry depends upon finding new fields where it may be profitably employed.

Paper No. 113

TESTING OF MATERIALS.

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STATIC TENSILE TESTS OF MATERIALS.

Modern research has shewn that the stresses to which parts of machinery and structures are subjected are so complicated and the behaviour of metals under different kinds of loading and mechanical and heat treatment is so variable that the ordinary static tensile test is insufficient for purposes of design, except for very simple cases. Even with simple cases of direct tensile stress, great care should be employed to see that a test on a short specimen in a tensile testing machine is really representative of the conditions to which the material is subjected in actual use. This has been well brought out in the results of a series of tensile tests on 15-metre lengths of hard drawn copper wire made by the Author, at the National Physical Laboratory, for the English General Post Office.[1] The testing machine, designed and constructed at the National Physical Laboratory, Teddington, is shewn diagrammatically in Figs. 1 and 2. The load is applied by water from the mains admitted to a straining cylinder (A), which will, with the water pressure available, produce a maximum pressure of 860 kilogrammes. The estimation of the load is made by a pendulum weight (B) revolving on ball bearings at the other end of the machine. The wires are held by grips attached to (a)

[1] Report on Hard Drawn Copper Wire published in Collected Researches of National Physical Laboratory, Vol. 8, 1912.

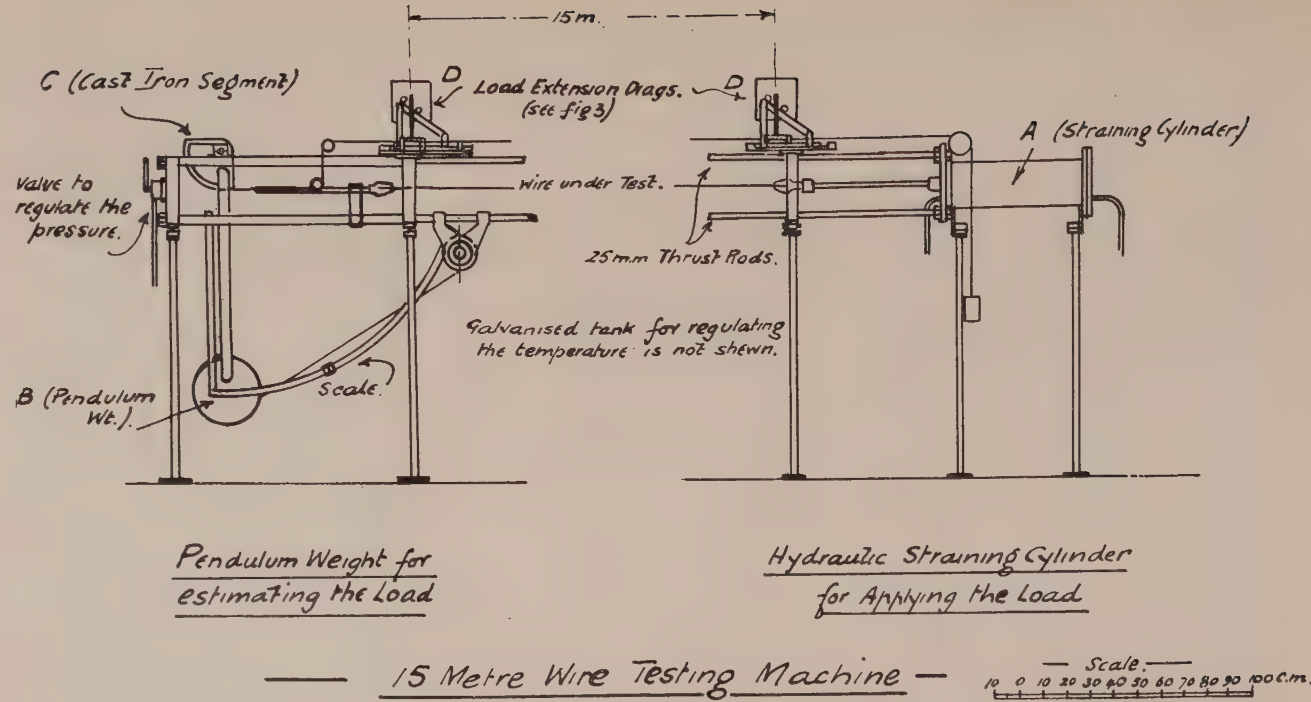
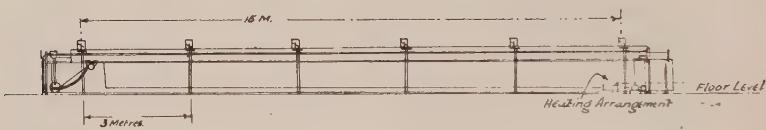


Fig. 1. Machine for Making Tensile Tests of Wire.

the piston of the straining cylinder, and (b) a flexible steel strip clamped to and taking the curvature of a cast iron segment (C) rotating with the pendulum, which thus has the load always applied to it horizontally.

Four 25 mm. diameter rods, stayed at intervals of 3 metres to prevent buckling, extend from the straining cylinder to the pendulum bracket, and these take the thrust instead of the machine being bolted to the floor. The specimen is supported horizontally in a galvanised tank, full of water, the temperature of which is regulated by circulation through a steam heater by means of a centrifugal pump. Provision is made (D) for taking six load extension diagrams simultaneously, one at every 3 metres of the wire. The apparatus for this is shewn in Fig. 3.



Arrangement of 15 Metre Wire Testing Machine.

Fig 2.

The diagram is drawn upon a vertical plate (E) [attached to a carriage (F)] by a pen (G) actuated by a clip (H) on the wire under test.

The carriage (F) is connected by a steel wire to the same segment (C, Fig. 1) as the flexible steel strip and, upon the application of the load, moves towards the cylinder at the same rate. There is, therefore, no relative movement between the carriage and the specimen grips at the pendulum end of the testing machine, and the horizontal component of any diagram drawn on the vertical plate represents the extension of the specimen.

The vertical plate moves down an inclined plane (J), at an angle of 45° to the horizontal, and the vertical component of the diagram is, therefore, equal to the horizontal movement of the carriage, which is proportional to the load. The diagram is thus a load-extension diagram.

In these experiments, the following points of interest were noted. (a) Very small "kinks", invariably present in the wire when used in long lengths, have a large effect on the breaking load and total elongation. The former may be 7% low and the latter 30% low in a wire containing a kink which would be considered inappreciable for practical use. (b) The temperature of testing is also important, it often depending upon whether the tests are conducted on a cold or hot day as to whether the wire passes or fails to reach the specified standard.

Between 20° C. and 60° C., a rise of 1° C. corresponds to a decrease of 0.1% of the breaking load and an increase of 0.3% in the total extension at 15° C. (Table I).

(c) Due to straightening out the wire after it has been in the coil, the "apparent" modulus of elasticity obtained by the first loading of a long length of wire is much lower than the actual modulus. The modulus also alters with the temperature (Table I), but this is only slight with the temperatures met with under climatic conditions, being about 0.0012% decrease for 1° C. rise of temperature[2]. A paper by Messrs. F. C. Lea, D. Sc., and O. H. Crowther, M. Sc.,[3] from which Table II is prepared, shews that at higher temperatures the rate of decrease of the modulus increases not only with copper alloys but also with steel.

FATIGUE TESTING.

In British engineering research laboratories, during the last 15 years, considerable attention has been paid to the behaviour of materials under the action of fatigue. It has been found that in the ordinary Wöhler test, by means of a

[2] A paper by H. Walker on "Variation of Young's Modulus Under an Electric Current", published in Proceedings of R. S. Edinburgh, No. 27, 1907, in which the temperature was raised by an electric current and shews the same decrease in the modulus with rise of temperature at temperatures below 100° C.

[3] "The Change of the Modulus of Elasticity and Other Properties of Metals with Temperature", by Prof. F. C. Lea, D. Sc., and O. H. Crowther, M. Sc., read before Section G of the British Association in America. Published in Engineering, October 16, 1914, page 487.

rotating loaded cantilever, it is possible to increase the speed of rotation up to 3,000 revolutions a minute without affecting the limiting resistance; so that the prediction of the safe range of stress in any material is no longer the tedious affair that it was previously.

Experiments, by electrical methods, up to rates of alternation of 7,000 a minute have been made by Kapp[4] and Hopkinson[5], but it appears that at these high speeds the results obtained are not of value for the purposes of the design of relatively slow moving parts of machinery. As regards the connection between the limiting ranges of stress for materials under alternating stress and the ranges of the elastic limits, in the investigation of which Bauschinger made his well known researches, Mr. Bairstow, in an important paper[6], has made experiments, the results of which constitute the first strong experimental support of Bauschinger's hypotheses that the range of stress between the superior and inferior elastic limits in iron and steel is the same in magnitude as the maximum range of stress which can be repeated without limit in a specimen of the same material without causing fracture. In the testing machine used by Mr. Bairstow for the purpose of the experiments, cyclical variations of direct stress were automatically produced at the rate of 2 per minute in such a manner that the extensometer used, which was of the Martens mirror type, was fixed to the specimen throughout the whole of the fatigue test, and in this way the whole history of the progress of fatigue was observed.

When the limits of stress were tension and compression of equal values it was found that, when the range of stress was above a definite value, the stress-deformation curve formed a closed loop, which was called the hysteresis loop, consisting of two parallel straight lines, corresponding to the variation of

[4] "Alternate Stress Machine", by G. Kapp, Zeits. Vereines Deutscher Ing., Aug. 26, 1911.

[5] "High-Speed Fatigue Tester", by B. Hopkinson, Proc. R. S., A. 86, 1912.

[6] "Elastic Limits of Iron and Steel Under Cyclical Variations of Stress", by L. Bairstow, A. R. C. Sc., Phil. Trans. R. S., London, Series A, Vol. 210.

R. S. = Royal Society.

stress from the limits of stress towards the mean stress, and two curved portions, corresponding to variations of stress from the mean value to the extreme values (Fig. 4). The width of this loop, which was the permanent "set" of the specimen per cycle, increased as the range of stress increased, but for a definite range of stress tended to a limit which was not greatly exceeded by subsequent repetitions of loading, even when this was the range at which fracture under fatigue eventually took place. Under these conditions of stress, the mean length of the specimen remained constant.

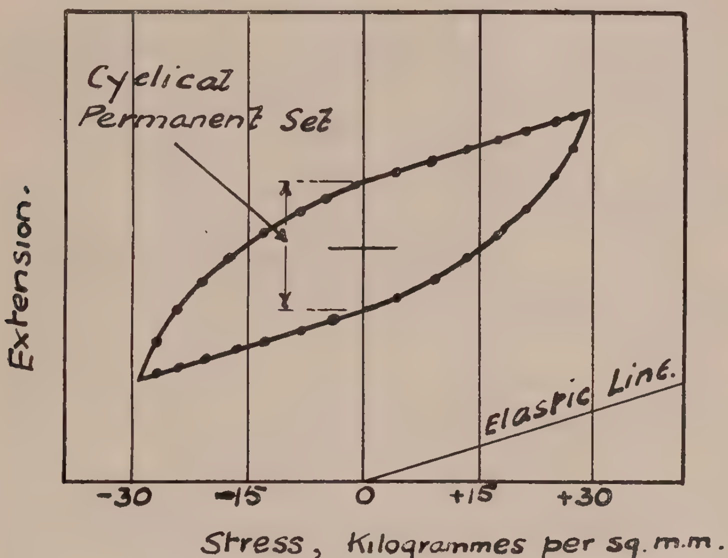


Fig. 4. A Typical Hysteresis Loop.

When the limits of stress were unequal the hysteresis loop was formed as before but was not closed, owing to the fact that the mean length of the specimen gradually changed because of the continued repetition of the same cycle of stress, i. e., the change of mean length of the specimen per cycle was the amount by which the hysteresis loop was enclosed. The amount of the permanent extension during the earlier stages of the breakdown becomes considerable as the superior limit

of stress approaches the static yield point, and if its value, after the first considerable stretch has occurred, be plotted against the corresponding values of the superior limit, it will be found that the curves will gradually come into coincidence with the ordinary static "force-elongation" curve at the "yield point". This is shewn in Fig. 5.

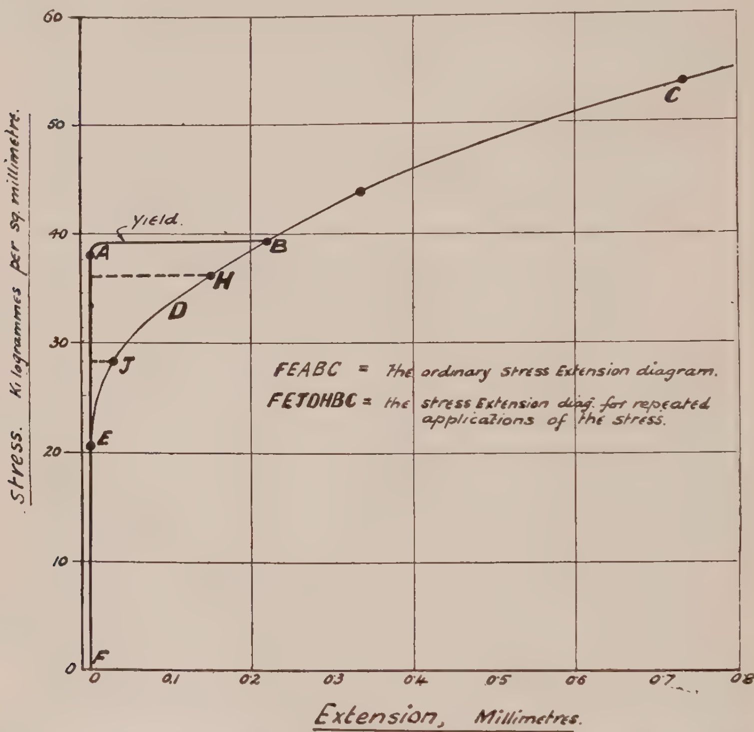


Fig. 5. Load Extension Diagram for Repeated Applications of the Range of Stress.

As Mr. Bairstow's method of determining the readings of the mirror extensometer is somewhat novel and has not been previously described, particulars of it are given in Appendix I.

The principal conclusions arrived at by Mr. Bairstow may be summarised thus:

(a) The "natural elastic range" is the value to be used in design, and, with equal compressive and tensile stresses, this value is identical with the "Wöhler safe range".

(b) The natural elastic range depends upon (1) The material and, (2) the lower limit of stress. The elastic range when the lower limit is 0 is less than that with equal tensile and compressive stresses (about 15% with axle steel and 6% with Bessemer steel).

(c) If a stress-extension curve is plotted, the extension being the value of the permanent extension reached after repeated alternations, it assumes the form found with hard drawn copper wire, which has no yield point but corresponds with the curve of Fig. 5 produced back to the "natural" elastic limit.

(d) Recovery due to stoppage of the alternations of stress is appreciable, being somewhat rapid for some materials. Recovery reduces the permanent extension at a given load and can be greatly accelerated by immersing in boiling water for a few minutes, as suggested by Muir.[7]

Some interesting examples of the results of recent fatigue tests made at the National Physical Laboratory are given in Table III and shew:

(a) Two aluminum alloys (Nos. 6 and 8) with the breaking strength of mild steel, but with a natural elastic range of half that of the steel.

(b) The effect a patent treatment may have on steel (Nos. 4 and 5); the breaking strength being reduced only 3½%, while the "natural" elastic range is reduced by 47%. Comparison between these figures is not quite fair, as the tensile specimen was solid and 1.4 cm. dia., while the fatigue specimen was a tube 0.11 cm. thick. The treatment is only for the surface and is used for thin plates; the fatigue test under these conditions gives the true percentage of reduction of strength with repeated loading due to the treatment, while the tensile test still represents, largely, the untreated material.

Two of the principal objections to the commercial adoption of the fatigue test are:

(1) Its cost. Most machines require from 4 to 6 test pieces in order to find the "elastic range", and some of these

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[7] "The Overstraining of Iron by Tension and Compression", by James Muir, D. Sc., M. A., Royal Society Proceedings, Vol. 77, pages 277-

have to be subjected to a considerable number of reversals of stress.

(2) The apparent variation in the endurance strength of test pieces cut from the same bar.

A recent report by Mr. C. E. Stromeyer [8] proposes a method of determining the elastic range on one specimen which can be carried out in one or two hours. He finds that when the natural elastic range is reached there is a sharp rise in the temperature of the test piece, and by gradually increasing the range he fixes this point. The test piece is surrounded by a loosely fitting India-rubber sleeve and through this a stream of water is passed. The temperature of the water before and after reaching the specimen is observed, and when a difference of 0.01° C. is noticed the elastic range is assumed to have been reached. It seems, however, as if Mr. Stromeyer determined the range by the accuracy with which his thermometers could be read (which is not necessarily the accuracy of the experiments) and before accepting the method independent evidence is required.

There is evidence that commercially there is an increasing demand for the alternating fatigue test. Several types of machine are in use (Appendix 2), but the rotating beam or cantilever (Wöhler) type seems to be generally preferred, and the Author has obtained reliable results from machines of this type, using hollow specimens and 2,200 alternations per minute.

COMBINED STRESS TESTS.

The stresses so far dealt with in this report have been simple stresses, but the great majority of cases met with in practice consists of varied combinations of stress with or without shock. In some of the machines for testing materials in alternating simple tension and compression, errors in the results are attributed to superimposed stresses, which are present owing to the design or workmanship of the machine, such as the thrust or pull not being axial and causing an additional

[8] Manchester Steam Users Association. Memorandum by the Chief Engineer, for the year 1913.

torque or bending moment on the specimen or a vibration of the machine or test piece.

Recently, the failure of a series of petrol engine shafts has been brought to the notice of the Author. These shafts have been designed for combined bending and torsion according to present practice, but neither the static tensile tests or impact tests (Table IV) shew adequate cause for failure. All the failures have fatigue fractures and the material has evidently failed under combined stress in fatigue. Cases like this are numerous and shew the demand for more information upon the question of combined stress in fatigue.

Practically all the available information is confined to combined stress under "non-repeated" loading conditions, and even research on this point is still in its infancy.

Three theories have been proposed to explain how a ductive material fails under any system of stresses, viz.:

(1). Greatest stress theory.

The material yields when one of the principal stresses reaches a certain amount. (The stress determined by a simple tension experiment.)

(2). Greatest strain theory.

The material yields when the maximum strain reaches a certain amount. (The yield-point strain in simple tension.)

(3). Greatest shearing stress theory.

The condition for initial yielding of a uniform ductile material is supposed to correspond to the existence of a specific shearing stress, the intermediate principal stress being without effect.

The latter theory was propounded in 1900 by J. J. Guest [9] and has received a great deal of support. This theory, now known as "Guest's Law", has been approximately verified by W. Scoble in 1906 [10] and W. Mason in 1909.[11] The yield stress

[9] J. J. Guest on "Strength of Ductile Materials Under Combined Stress", Proceedings of the Physical Society, Vol. XVII, 1899-1901, page 202.

[10] Walter Scoble on "Strength and Behaviour of Ductile Materials Under Combined Stress", Phil. Mag., Dec., 1906, Vol. XII, page 533.

[11] W. Mason on "Mild Steel Tubes in Compression and Under Combined Stress", Proceedings I. M. E., 4, 1909.

was taken as the point of failure, this being no doubt due to the method of testing, which did not allow of the tests being continued until failure.

The most important case, in practice, of combined stresses, with ductile materials, is that of bending and twisting. For this it is usual to design for an equivalent bending moment or equivalent twisting moment.

Under the three theories we have:

$$\text{Equiv. bending moment} = \tfrac{1}{2}\left(M + \sqrt{M^2 + T^2}\right) \quad \text{(Greatest stress theory).}$$
$$= \tfrac{3}{8} M + \tfrac{5}{8}\sqrt{M^2 + T^2} \quad \text{(Greatest strain theory).}$$
$$= \sqrt{M^2 + T^2} \quad \text{(Greatest shearing stress theory).}$$

The two former give sizes which are too small.

Guest in conclusion says: "The specific shearing stress is better determined by taking one half of the tensional yield-point stress than from the results of torsional experiments in which the sharpness of the yield-point is masked".

IMPACT TESTING.

Mr. Hatt, in 1904 (91)*, as a result of some experiments in dynamic tension, came to the conclusion that for steels there is little difference in the total elongation and the unit work in fracture, whether the fracture is brought about in ten minutes or 1/100 of a second.

Messrs. Stanton and Bairstow (168), in 1908, and Delikhow (43), in 1909 (using a drop-hammer type of impact machine), have also shewn that the tensile impact test (i. e., dynamic tensile test) gives numerical values which agree with the results of the static tensile test. The Author recently (with a pendulum impact machine) on steel and aluminum alloys obtained further confirmatory evidence. All these tests shew that the dynamic tensile test can take the place of the static tensile test, with the exception of the determination of the primitive elastic limit, which, as is well known, is an arbitrary limit depending on the mechanical work put into the material. Not only is there little difference in the total extension and contraction, but the total work done in fracture is the same,

* These numbers refer to the Bibliography, Appendix III.

and is given directly as the result of the test without any laborious calculation from a load extension diagram. Another point, important in some cases, is that the testing plant required is considerably cheaper and can be used for notched-bar tests, mentioned in the following paragraphs.

In 1909, Charpy (33), in a report on impact tests of metals, shewed that a static tension test is more efficaciously supplemented by a notched-bar bending test than by any other test, as the latter test gives information regarding the fragility of the material which other tests do not furnish. Milton (124), Sankey (152), and other experimenters have come to the same conclusion. Stanton and Bairstow (168), by giving repeated alternating blows of a tup to a notched specimen (in bending), came to the conclusion that when the number of blows to break the specimen is small, the relative resistances to shock are in agreement with single-blow bending impact-tests, but when the number of blows is considerable, the relative resistances are practically reversed, i.e., the materials which are weakest under the single-blow test become the strongest. Also under the latter conditions the resistance is proportional to $f^2/2E$ where f is the "natural" elastic limit and E is the modulus of elasticity. These conclusions agree with the results given in Table IV, which shews that the resistance to single-blow bending impact is low when the elastic limit is high and vice versa.

Thus it is evident that single-blow bending impact-tests on notched specimens reveal in an easy and unmistakable manner the power of resistance of materials to conditions which the static tensile test does not. Mr. Charpy (33) has shewn that materials which give good results with this impact test stand well in practice, whereas, the same class of materials giving low values fail, although the result from the static tensile test in both cases is the same.

The standardisation of the notched-bar impact test has received serious consideration since 1906, especially by the International Association of Testing Materials. It was soon found that only machines employing the single-blow method of fracture could be expected to furnish comparable results, but the law of similarity between specimens of different sizes has not yet been established. Special experiments are being under-

taken in order to determine this law (196), and, in the meantime, two types of specimens have been provisionally adopted, viz:

1. 30 x 30 x 160 mm. specimen, notched to depth of 15 mm., and bottom of notch cylindrical, of 2 mm. radius. Distance between supports, 120 mm.

2. 10 x 10 x 53.3 mm. specimen, notched to depth of 5 mm., and bottom of notch cylindrical, of 2/3 mm. radius. Distance between supports, 40 mm.

Both Blount (14) and Harbord (88), as a result of a series of impact tests on notched specimens on different machines, concluded that duplicate tests on the same material shewed considerable variation.

Frémont (63) and Delikhow (43) both say that too much attention is given to obtaining a large size of specimen which will repeat results, as these do not always expose the brittleness with the same rigour as small specimens.

The Author's experience has been that, with specially selected materials, the results from impact tests shew but little variation and that variation in the results is due to differences in the material under test. A brittle material or brittle places in a material can easily be identified.

HARDNESS AND ABRASION TESTING.

Hardness can be considered from two different standpoints, viz, as resistance to abrasion or resistance to indentation. Some experimenters consider it from both aspects, but most of them from only one point of view. The following definitions have been published:

A. Hardness is the resistance to indentation. [Ballentine (8)* and Turner (180).]

Hardness is the resistance offered by a solid substance to the entry of another substance into it. [Wahlberg (184).]

B. (1). Hardness is the property whereby a body is enabled to blunt or wear away the edge of a tool used upon it. [Turner (177).]

(2). Hardness is the resistance offered by a smooth surface to abrasion,—Dana's definition. [Brynton (20).]

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* These numbers refer to Bibliography, Appendix IV.

Instruments have been designed and used to determine the hardness of materials as suggested by each of the above definitions.

A. The indentation method has been applied in three ways:

(1). The material tested with itself.

Réaumur, in 1722, (142) pressed together two right angled pieces of the material. The depth of indentation formed a measure of the hardness.

Foeppl, [see Wahlberg (184)] placed two cylinders of the material with their axes at right angles and pressed them together. The hardness was equal to the pressure per unit of flattened surface.

(2). The material tested by pressing (statically) a harder material into the material under test.

Ordnance Dept., U. S. A. (126), Hardness = Vol. of indent produced by pyramidal point under given load.

Calvert and Johnson (23), H = Weight indenting a truncated cone into material to a given depth in a given time.

Hertz (56).

Middleberg (117). A knife edge used as indenting tool.

Auerbach (5).

Kirsch (75). H = Weight indenting a cylindrical plunger into material, permanently to a given depth.

Unwin (183). Square bar indenting tool.

Brinell (17 and 18). Ball as indenting tool.

Ludwig (94 and 95). Cone as indenting tool.

(3). Indentation produced by Impact. Ballentine (8), Brinell (18), Kerner (74), Martel (100), Guillery (206), and Shore (162); the latter measuring the height of rebound.

It has been shewn by numerous observers (Ast, Breuil, Brinell, Charpy, Dillner, Kurth, Le Chatelier, Leon, etc.) that the indentation method is really a strength test; the hardness number given by it being proportional to the tensile strength. It is, therefore, a simple means of comparing tensile strengths, provided the material is homogeneous.

Each method employed has its limitations:

(a) The Shore scleroscope results are affected by the shape of the small hammer.

(b) The Ludwig cone test is not suited for very hard materials.

(c) The Brinell test gives results varying with the size of ball, load, etc.

For indentation hardness tests, the Author uses (a) and (c), modifying the Brinell number, as suggested by Benedicks (12), in order to, as far as possible, overcome the size of ball and load difficulty.

$$\text{Brinell Hardness No.} = \frac{\text{Load}}{\text{Spherical area of indentation}}.$$

$$\text{Benedicks Hardness No.} = \frac{\text{Load}}{\text{Spherical area of indentation}} \times 5\sqrt{\text{Radius of ball.}}$$

(All measurements in kg. and mm. units.)

The method suggested by Moore (119) as a result of Meyers (111 and 112) conclusions, viz, that "the Hardness Number with the ball test should be the mean pressure per unit area when the diameter of the impression is half the diameter of the ball", is a satisfactory method of overcoming the ball test difficulty but it is too complicated to apply in ordinary works practice as it needs two impressions with the same ball and different loads.

B. The abrasion method has been applied in 3 ways:

(1) As a scratch test; usually the load on a diamond of given angle to produce a scratch or a scratch of standard width. As exponents of this method we have Réaumur, 1722, (142); Mohs, 1822, (118); Seebeck, 1833, (159); Turner, 1886, (177); Martens, 1888, (102); Parsons, 1910, (131); and Cohen (26).

(2) By grooving or drilling with a hard steel tool or diamond. Used by Bottone, 1873, (14); Plaff, 1883, (134); Haussner, 1892, (53); Jagger, 1897, (67); Keep, 1900, (69).

(3) By wearing away with or without an abrasive.

(a) Pure abrasion, such as grinding on an emery wheel or rubbing with an abrasive powder.

Jannetaz and Goldberg, 1895, (65); Behrens, 1895, (10); Rosiwall, 1896, (154); Robin, 1910, (151); Stough-

ton and Macgregor, 1911, (173); Pagé, 1913, (130); Stanton and Batson, 1913, (170); Gary, 1904, (40); and Warren, 1911, (186) who used a sand blast.

(b) Lubricated sliding friction. The metal subjected to wear by contact with a moving metal surface well lubricated. This method was used by Derihon (205).

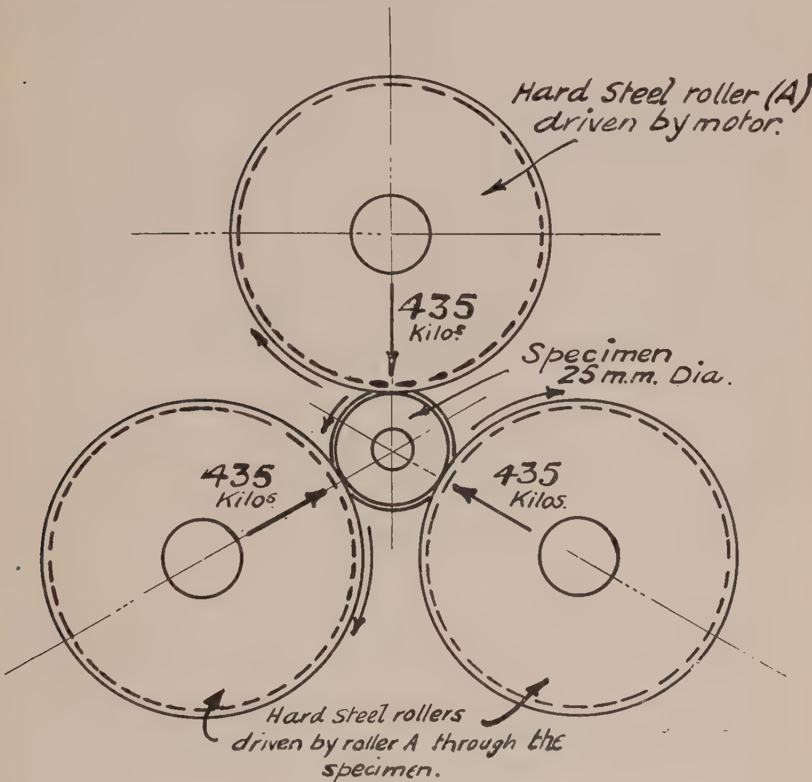

Fig. 6. Method of Making Abrasion Tests.

(C). Dry friction. Used by Saniter, 1908, (157 and 158) and Stanton, 1908, (169).

It has been found by Greaves (44) and Turner (177) that the scratch method and the drill method agree very well with the indentation method, but experimenters on the wearing away test do not find the same agreement. They find that some ma-

terials which give a low value from the indentation method have a high value in the wear method, and their results agree with practical experience of these materials under wear [Saniter (158), Devries (29)].

Some dry rolling abrasion experiments were recently made by the Author on Stanton's machine (169), in which the test piece is placed between three hardened tapered rollers mounted at 120°, as illustrated in Fig. 6. The top roller is driven by a motor through a flexible coupling, while the under rollers are mounted on friction rollers. The results on a series of rail-steels, together with the ball test results, are given in Table V. These, together with the results of other experiments, shew that a wear test is quite distinct from the indentation test and gives results which are of practical importance.

The Author desires to express his thanks to Dr. Stanton, F. R. S., M. Inst. C. E., and Mr. Leonard Bairstow, A. R. C. Sc.. for valuable assistance in the preparation of the paper.

APPENDIX I.

MEASURING ARRANGEMENT FOR MIRROR EXTENSOMETERS.

This is shown diagrammatically in Fig. 7. The telescope (T) is attached to a support (S). Immediately under the telescope, and attached to the same support, is a platform (P) whose height from the ground can be quickly adjusted. It can be locked in any position by a knurled-headed screw (A). The telescope attachment is kept in contact with the platform by means of a spring (B), and their relative position in a horizontal plane can be accurately adjusted by means of the pivoted nut and screw (C).

The platform carries an illuminated scale (D), two fixed mirrors (M_1 & M_2), and two mirrors (N_1 & N_2) pivoted vertically so that their position can be altered by means of a screw and spring (E).

The illuminated scale is reflected from one of the "Martens" rotating mirrors on to the pivoted mirror N_1, and the angle of this is adjusted to bring the reflection into the telescope by means of the fixed mirror M_1. The other "Martens" mirror is made to reflect the illuminated scale on to the pivoted mirror N_2, and this is adjusted to bring the reflection into the same telescope. The difference between the lengths of the two rays is so small that the telescope can, at the same time, focus the scales reflected by each of the Martens rotating mirrors. The mirrors reflect "ghosts", and these are cut out by inserting a mask (Fig. 8) in the eye piece of the telescope and by having adjustable brass strips (R. Fig. 7) fixed in between the mirrors on the platform and the extensometer.

The mirrors used must be absolutely flat and great care should be taken to see that they are not distorted when they are clamped in position.

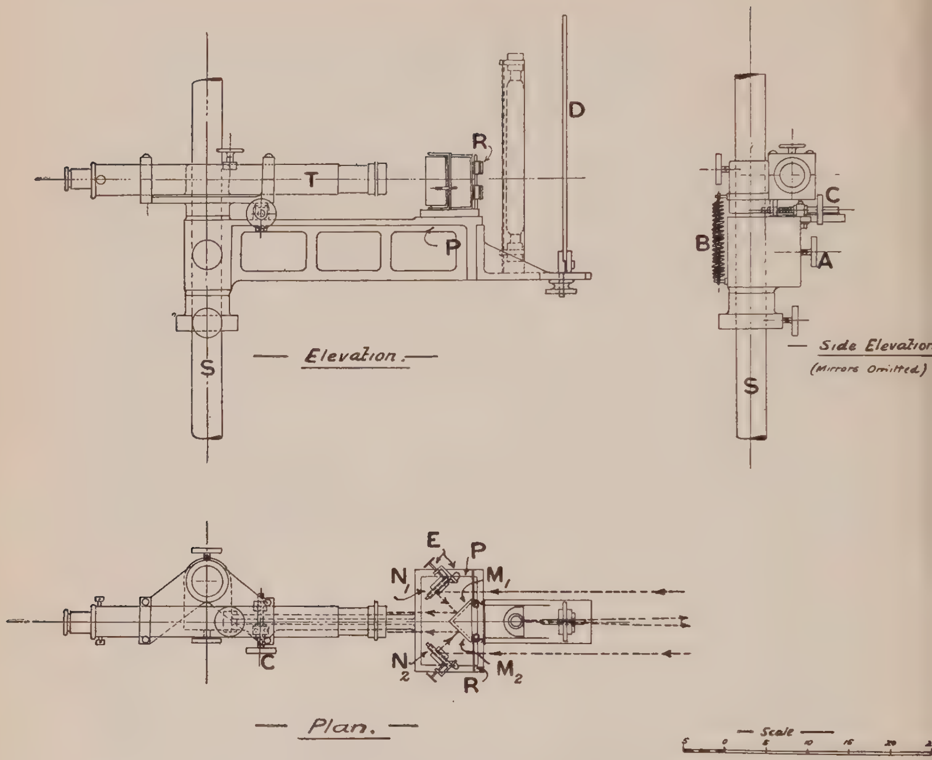

Fig. 7. Measuring Arrangement for Mirror Extensometer.

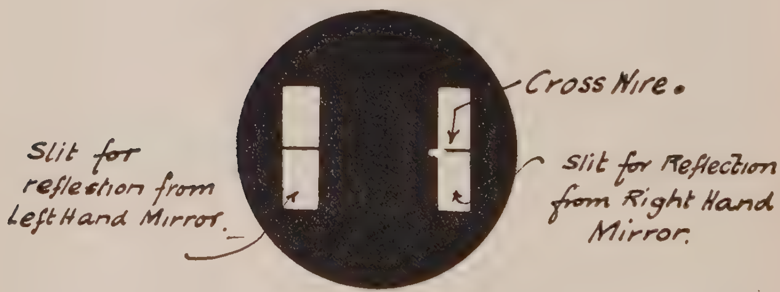

Fig. 8. Mask in Eye-piece of Telescope of Measuring Apparatus for Mirror Extensometer.

APPENDIX II.

Machines Used for Testing Materials Under Alternating Simple Stresses.

A—Rotating Specimen Machines.

All accounts marked * give a description of the machines used.

1 Rotating Bar or Wöhler Type, for Repeated Bending in Opposite Directions.

1 Wöhler, 1871*, Unwin's "Materials of Construction". Specimen was a rotating cantilever loaded at the end by a spring balance (20-25 r. p. m.)

2 Sondericker, J., "Quarterly Journal", 1892, of Massachusetts Inst. of Technology, "Repeated Stresses". A rotating bar having a short length under constant bending movement. (350-500 r. p. m.)

3 Stead & Richards, Journal of Iron & Steel Inst., 1903, II. Wöhler type.

4 Dudley, C. B., 1904, Feb., 1914, Iron & Steel Metallurgist, "Alternate Bending Stresses". Wöhler type.

5 Gardener, J. C., "Effect of Stress Reversal on Steel", Journal Iron & Steel Inst., 1905. Wöhler type.

6 Rogers, F., "Heat Treatment & Fatigue of Steel", Journal of Iron and Steel Inst., 1905. Wöhler type.

7 Unwin, W. C., 1905, "Expts. on Rotating Bars at Different Temperatures", Proc. Inst. Civ. Eng., CLXVI. Wöhler type.

8 Howard, J. E.,* "Alternate Stress Testing", Engineering Record, Sept. 22, 1906. Sondericker type (500 r. p. m.).

9 Eden, E. M., Rose and Cunningham,* "The Endurance of Metals", Proc. Inst. of Mech. Eng., 1911-4, pg. 839. Sondericker type (250-1300 r. p. m.).

10 Stanton & Pannell,* "Experiments of Strength and Fatigue Properties of Welded Joints," Proc. Inst. Civil Eng., 1911, Vol. 188. Wöhler type (2200 r. p. m.).

11 Roos, J. O.,* "Some Static and Dynamic Endurance Tests", Int. Assoc. Test'g Materials, Paper V, 1912. Wöhler type (1200-2400 r. p. m.).

12 Stromeyer, C. E.,* Manchester Steam Users Assocn., Memo by Chief Eng. for Year 1913. Wöhler type. Fatigue range fixed calorimetrically.

13 Marten, A., ''Fatigue Bending Tests''. Made between 1892 and 1912, Science Abstracts, 1914, No. 1371. Wöhler type. During the test a rough estimate was made of the rise in temp. at the centre of the bar. Considerable heating—fracture less than 10^6 reversals. Slight heatings;—number of reversals before fracture considerably increased.

2 **Other Types of Rotating Specimen Machines.**

1 Stanton, T. E.,* ''New Fatigue Test for Steel'', Journal Iron and Steel Inst., 76, 1908. Hollow ring of rectangular section rotates between three hardened rollers symmetrically situated. The upper roller being loaded, .3 cycles per rev. of specimen. (2250 cycles per minute.)

B—Direct Stress Machines.

1 Wöhler, 1871,* Unwin's ''Materials of Construction''. Repeated tensions obtained on a fixed vertical specimen by means of a system of levers worked by a crank and connecting rod.

2 Bauschinger, 1886, Unwin's ''Materials of Construction''. Similar to Wöhler's machine.

3 Coker, E. G., 1898,* ''Endurance of Steel Bars Subjected to Repetitions of Tensional Stress,'' Proc. Inst. Civil Eng., Vol. 135. Wicksteed type of testing machine loaded up from 0 in tension. Extensions measured by Kennedy extensometer. (Repeated loadings by hand.)

4 Reynolds & Smith,* ''A Throw Testing Machine for Reversals of Stress,'' Phil. Trans., A, 199, 1902. Weight supported vertically by specimen to be tested, the upper part of which receives a periodic motion due to a crank and connecting rod—Inertia of weight gives alternate tension and compression. (1300-2500 r. p. m.)

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$$H = \frac{16 \times P\, D^{n-2}}{\pi\,(2d)^n} \qquad n = \frac{\log P_1 - \log P}{\log d_1 - \log d}$$

H = hardness no.
P = load in kilograms.
D = dia. of ball
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TABLE I.

Dia. of Wire		Temp. °C.	Breaking Load		Total Extn. on 50 ft. (1524 cms.)*		"Apparent" Modulus of Elasticity	
Ins.	m.m.		Lbs.	Kilograms	Ins.	m.m.	Lbs./sq. in.	Kilos. per sq. m.m.
.158	4.01	49.5	1166	529	4.68	119	$10^6 \times 12.9$	$10^3 \times 9.1$
		15.5	1200	544	4.65	118	$\times 13.3$	$\times 9.3$
		3.3	1210	549	4.48	114	$\times 13.5$	$\times 9.5$
		—11.6	1240	562	4.71	120	$\times 13.9$	$\times 9.8$
.137	3.48	61.3	873	396	5.10	130	$10^6 \times 13.0$	$10^3 \times 9.1$
		17.5	908	412	4.79	122	$\times 12.9$	$\times 9.1$
		2.8	924	419	4.62	117	$\times 14.2$	$\times 10.0$
		—12.2	945	429	4.52	115	$\times 14.6$	$\times 10.3$
.112	2.84	60.1	578	262	4.17	106	$10^6 \times 13.1$	$10^3 \times 9.2$
		16.7	610	277	4.20	107	$\times 13.9$	$\times 9.8$
		0.3	620	281	4.58	117	$\times 14.0$	$\times 9.8$
		—10.9	632	287	4.58	117	$\times 14.2$	$\times 10.0$

* Total Extension = Elastic Extension and Permanent Extension.

TABLE II.

Mild Steel			Nicro Copper		
Temp. °C.	Modulus of Elasticity		Temp. °C.	Modulus of Elasticity	
	Lbs./sq. in.	Kilos. per sq. m.m.		Lbs./sq. in.	Kilos. per sq. m.m.
0	$10^6 \times 29.5$	$10^3 \times 20.7$	0	$10^6 \times 11.2$	$10^3 \times 7.9$
270	$\times 28.0$	$\times 19.7$	100	$\times 11.0$	$\times 7.7$
375	$\times 26.0$	$\times 18.3$	380	$\times 9.0$	$\times 6.3$
495	$\times 20.0$	$\times 14.1$	650	$\times 6.0$	$\times 4.2$
610	$\times 12.0$	$\times 8.4$			

TABLE III.

Material	Static Tensile Test.					Natural Elastic Range Kg./m.m.^{2}
	Primitive Elastic Limit Kg./m.m.^{2}	Yield Stress Kg./m.m.^{2}	Breaking Stress Kg./m.m.^{2}	% Extension on 50.8 m.m.	Modulus of Elasticity Kg./m.m.^{2}	
1. Steel	32.4	33.2	49.1	42.0	$10^3 \times 21.4$	± 23.6 (47.2)
2. Steel	30.9	36.5	63.6	26.0	$10^3 \times 21.2$	± 29.5
3. Steel	......	39.5	70.6	on 25.4 m.m. 23.0	....	± 25.2
4. Mild Steel (Untreated.)	......	38.0	48.5	on 50.8 m.m. 36.0	....	± 28.4
5. Mild Steel (No. 4, surface of specimen specially treated.)	......	36.9	46.8	36.5	....	± 15.0
6. Aluminium Alloy	21.3	39.5	47.3	16.0	$10^3 \times 7.4$	± 11.8
7. Aluminium Alloy	18.1	29.1	34.5	14.0	$10^3 \times 7.2$	± 10.2
8. Aluminium Alloy	26.8	39.5	49.2	13.5	$10^3 \times 6.8$	± 11.8
9. Magnesium Alloy	4.4	9.9	27.2	9.0	$10^3 \times 4.6$	± 7.9

TABLE IV.

Tests on Crankshafts.

No. of Shaft	Static Tensile Test						Single Blow Impact Test (Izod) Energy absorbed in fracture Kg. metres.
	"Primitive" Elastic Limit Kg./m.m.^{2}	Yield Stress Kg./m.m.^{2}	Breaking Stress Kg./m.m.^{2}	Modulus of Elasticity Kg./m.m.^{2}	Extn. on 25.4 m.m. %	Red'n of Area %	
1	71.8	90.1	102.1	$10^3 \times 20.5$	13.0	24.6	.50
2	66.9	81.7	96.1	$\times 20.7$	17.0	31.4	.61
3	57.0	77.6	87.6	$\times 20.5$	20.0	46.1	1.05
4	35.3	54.3	71.3	$\times 20.5$	23.0	43.0	1.69

TABLE V.

Abrasion and Ball Indentation Tests.

No. of Test Piece	Abrasion Test	Ball Indentation Test	
	Loss of Wt. in Milligrammes (Load = 435 kg.) (No. of revs. of specimen = 8250)	Ball = 9.525 mm. dia. Load = 3000 kg.	
		Brinell No.	Benedicks No.
1273	32.8	212	289
1031	21.4	212	289
1029	5.9	214	293
1030	9.5	228	311
1272	16.4	234	319
1079	9.7	234	320
1027	6.7	237	323
1034	7.6	258	352

Paper No. 114

TESTING FULL SIZE MEMBERS.

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INTRODUCTORY.

At the Sixth Congress of the International Association for Testing Materials, held in the United States, in the year 1912, a paper was presented by the writer, a portion of which was devoted to "The Progress in the United States in Testing Full Size Pieces".

It contained a list of most of the laboratories in this country which had, as a part of their equipment, large testing machines, whether owned or controlled by the Government, by industrial corporations, by educational institutions, or by private companies; also a very brief summary of the tests of full size pieces that had been or were being performed by each. The object in view was to outline the progress thus far made, and the activity along these lines existing at the time.

It is not the purpose of the present paper to give or to discuss these statistics; but, starting with such an outline in retrospect, to consider present and future needs for work of this character, and the requirements to be observed to render the results of most value.

The greater portion of my remarks will therefore be devoted to a consideration of the following topics, viz.:

(I) The meaning and scope of the term "Testing Full Size Members".

(II) The importance of making such tests, and the reasons therefor.

(III) The need for their performance on a systematic and well organized plan, so that the results obtained may be of the greatest value in engineering practice.

(IV) An enumeration of the kinds that have thus far received the most attention.

(V) A summary of those most needed, including a consideration of certain classes concerning which comparatively little, and of others concerning which no progress has been made thus far.

(VI) Consideration of the value and possibilities of co-operation.

I. THE MEANING AND SCOPE OF THE TERM "TESTING FULL SIZE MEMBERS.

The term "testing full size members" is a convenient expression employed to denote the testing of members of a structure or of a machine of the size and dimensions actually used, and in so arranging the experiment (whether it is made with the member in place or not), as to subject it as nearly as possible to such stresses as it would have to bear in actual use.

In this paper it will be considered to include, also, the making of any auxiliary tests, whether physical or chemical, that may be necessary to determine the quality of the metal.

Observe moreover that a full size member of a bridge or of a building is of very different dimensions from one used in an automobile or an aeroplane; also that the method to be pursued must vary in consequence of the differences in the nature of the stresses to which the members are subjected in practice, or in consequence of differences in the construction of the full size members.

By way of illustration note that, in the case of large timber beams and columns, the effects of knots, cracks, crooked grain, and other defects are to be reckoned with to such an extent that they form the principal factor in determining the strength of the beam or column, and that similar remarks may be made regarding the effects of blow holes, and of imperfections in the case of cast iron columns.

II. IMPORTANCE OF TESTING FULL SIZE MEMBERS.

In the design and construction of buildings, bridges, machinery, and other engineering structures, many cases arise where the determination of the proper forms and dimensions

of certain pieces, when based only upon the usual theoretical formulae, methods, etc., together with the results of such tests as are usually made upon small specimens, to ascertain the chemical and physical properties of the materials, is not satisfactory, because it leaves out of account certain conditions depending on either the details of the construction or the characteristics of the material as it exists in the full size piece.

In some instances no theoretical formulae have been developed that will stand the test of actual practice, and the only formulae available are purely empirical, some of which have no reasonable basis, while in others the empirical formulae apply fairly well within a limited range, but fail to agree with the facts in many practical cases beyond that range.

Under such conditions as those mentioned above, and also in others, tests of full size members are of importance—

a To establish the constants for use when the formulae and theories are applicable.

b To determine whether the formulae and theories apply.

c To furnish a basis of fact for the development of new theories that are suitable when those in common use are not.

d When no theories can be developed that are applicable, to furnish a basis for the development of empirical formulae that are.

e When none of these things can be done, to supply actual knowledge as to the safety of the particular member.

In the absence of the resulting information the engineer can only proceed in some such way as one of the following, viz.:

a To base his design and construction upon past experience, if that is available and applicable.

b To do the best he can with the formulae, and results of tests that he can find, even though they do not strictly apply to his case.

c When any piece breaks, to add more metal at the point where the fracture occurred.

In regard to the first of these methods, if the experience available is sufficiently complete, it is of course a guide, as the knowledge that a certain member, having certain forms and

dimensions, and constructed in a certain way, of a known quality of material, has given satisfactory service is a matter to be taken into careful consideration in any event, and must necessarily have its influence on future practice; but in very many cases, especially where the industry is developing, the dimensions and proportions are different (often larger) and frequently the material used in the new member differs from that covered by the experience.

The second method is the one that should of course be pursued in all cases.

The third was often followed in olden times when engineering was not so far developed as it is at present; but it is now recognized not only as unsafe and illogical but also as liable to lead to unnecessary weight and expense.

The recognition of the importance of making tests of full size members had its inception at a period which began about the year 1875.

At that time tests of wrought iron bridge columns, and also of eyebars were made, and later of I beams, of small plate girders, of columns of cast iron, and of timber beams and columns, while in recent times many have been conducted in many places, of columns, beams, slabs, and floor systems of reenforced concrete.

In most of the tests of this character, more attention has been paid to the breaking strength than to any thing else, and but little has been done experimentally to study the effect upon the properties of the material, of the conditions met with in service, and to ascertain the distribution of the stresses in the different parts of a complicated structure, or in those of a built up member, during the erection of the structure, and later when subjected to the conditions of service.

It is only very lately that measurements have been made to find out the strains and stresses in the different parts of columns, eyebars, beams, etc., in buildings and bridges under erection; but the results obtained have been found so valuable that more and more attention is being paid to this class of investigations, and the amount of such testing is steadily increasing as regards such structures; and in all probability it will find application in many other cases.

III. THE NEED FOR THE PERFORMANCE OF SUCH TESTS ON A SYSTEMATIC AND WELL ORGANIZED PLAN, SO THAT THE RESULTS OBTAINED MAY BE OF THE GREATEST VALUE IN ENGINEERING PRACTICE.

While at the present time there are available in print the results of a large number of tests of full size members, and while the benefit already derived from them has been of great value and has contributed much to our knowledge along these lines, and hence to that of how to proceed to secure proper safety in our engineering constructions, nevertheless there have been but few cases in which the investigation was carried on with such completeness as to constitute a thoroughly comprehensive and well organized series.

It should be so planned that the effect of altering each of the variable features in the design and in the construction of the member may be determined experimentally, and also there should be made the usual tests upon specimens of the metal to determine the chemical, physical, and other properties of the material entering into the construction of the full size piece. Moreover we should also seek the effect of a variation in each of the above stated properties.

By way of illustration observe that while many experiments have been made upon the compressive strength of full size built up columns of the usual structural steel, that is, of carbon steel having a tensile strength of about sixty thousand pounds per square inch, a good ductility, and a yield point equal approximately to one half the ultimate, and while a few results have been obtained of tests of solid columns (such as connecting rods) made of carbon steel of about eighty thousand pounds per square inch tensile strength, and a tensile yield point equal to about fifty per cent of the ultimate, the writer knows of none upon the compressive strength of full size members, whether built up or solid, made of heat treated or of alloy steels, nor of any in the case of carbon steels where the tensile yield point (whether artificially raised or not) is very considerably more than fifty per cent of the ultimate tensile strength.

As to the first mentioned steels (carbon structural steels) the evidence available shows that for ratios of slenderness that do not induce buckling, the crushing strength per square inch

is approximately equal to the tensile yield point, but to determine whether the same conclusion applies to heat treated, to alloy steels, and to those having a very high tensile yield point as compared with the ultimate, experiments are needed.

Moreover results are available that give us approximately the ratio of length to radius of gyration, at which the crushing strength in built up columns of structural carbon steel begins to decrease on account of buckling, and we also have evidence to show that in the case of solid columns of carbon steel of about eighty thousand pounds per square inch tensile strength, the corresponding ratio is very much higher, but experimental evidence is lacking so far as the writer knows to fix the corresponding ratio in the cases of heat treated, of alloy steels, and of steels with an abnormally high yield point.

Moreover this and other information of a similar character is becoming of more and more importance since these last mentioned kinds of steel are coming more into use, especially in machinery and whenever lightness is required.

Among those that have been designated as "other properties" may be noted the liability to develop cracks, and other defects, including segregation and lack of homogeneity.

IV. A SUMMARY OF THE CLASSES OF FULL SIZE TESTS MOST FREQUENTLY MADE UP TO THE PRESENT TIME.

As already stated, the investigations of this character thus far made have been mostly such as have found their application in either buildings or bridges.

a Thus there have been performed a considerable number of tests of eyebars and of other tensile rods, and the fact has been made evident that inasmuch as the large members receive as a rule less working in the rolls than small ones, they possess less strength per square inch, and there are many results available as to the relation between the size, and the amount of the reduction in strength. On the other hand comparatively little has been done towards studying experimentally the effect of the proportions of the eyes, not only in eyebars but in other cases, upon their strength.

b Many tests have been made of full size, built up columns of wrought iron and of structural steel, but little has been done

by way of making an experimental study of the details, such as the stresses in and the effect of different spacing of the braces, and of the effect of the various methods in use for attaching the floor beams, especially as to the resulting eccentricity of the load thereby induced.

c Quite a number of tests have been conducted upon the strength of full size cast iron columns such as are used in buildings.

d Many tests have been made upon tolerably large tensile riveted joints, but in the majority of cases the rivets have been of wrought iron, while the number in which modern steel rivets have been used is much smaller. Also very little has been done by way of investigating experimentally the various riveted joints of members supporting transverse loads, such as the splices or the connections of beams to columns, girders, etc.

e There are published the results of a very large number of tests of various sorts upon beams, slabs, columns, and floor systems of reenforced concrete, but many more are needed.

Thus there is available comparatively little information based on experiment, of mushroom systems, of arches, dams, and walls including retaining walls of reenforced concrete.

f The number of full size tests of masonry walls, piers, arches, etc., whether of brick or stone, is very small, though some work has been done.

g Quite a number of tests have been made upon the strength of I beams and some on that of plate girders, but the results obtained are very far from furnishing all the information required in these directions.

h A large number of experiments have been performed upon repeated and alternate stresses, but this investigation is of great moment, and far more work of this character is requisite.

i While many impact tests have been made to determine whether or not axles, rails, etc., fulfill certain arbitrary conditions laid down in specifications, very little has been accomplished towards putting this matter on a practical and a scientific basis.

j Enough has not been done experimentally regarding the

strength of hooks, eyes, links, and of machine frames similarly loaded, and a great deal more is required.

k There have been some tests on the strength and distortions of flat and curved plates of different sections, with a view to their bearing upon the strength of the ends of pressure vessels, such as boilers, digesters, gas tanks, cylinder heads, and to some extent of pistons, but many more are required, not only of a greater variety but also on a larger scale.

V. A SUMMARY OF SOME OF THE KINDS OF TESTS OF FULL SIZE MEMBERS MOST NEEDED.

Any attempt to classify the tests which should be made, in a systematic way, according to the sorts of stresses, as Tension, Compression, Transverse, Shearing, Torsion, etc., would necessarily be attended with considerable difficulty, as in the majority of cases we have a combination, as tension and bending, bending and torsion, etc., or else different parts of the member may be subjected to different kinds of stresses, thus one part may be in tension and another in tension and bending combined, etc.

Therefore the most feasible method of procedure would seem to be to take up the different kinds of members individually and deal with them according to the parts they play in actual use. The investigation of each should be as complete as possible, not only as to the strength of the main body of the member, but also as to that of its end fastenings, its braces if any, etc.; all this should be done as far as possible under all the various conditions of loading to which the member is liable to be subjected in practice. Thorough tests should also be made to determine the chemical, physical and other properties of the material which enters into its construction.

In view of the above, notwithstanding the fact that there would be more or less overlapping, we will adopt as one broad division such members as are employed in building, and in bridges, and as a second, such as are used in pressure vessels, and in various kinds of machinery.

The investigation of those belonging to the first of these two divisions has heretofore received the most attention on the part of the investigators; and in regard to them I shall first

enlarge upon some of the remarks already made in this connection as to the additional work that should be done, and also call attention to some other kinds not already mentioned.

Taking up first therefore the kinds of tests of full size members already enumerated:

a **Eyebars and other Tension Members.** The importance of studying experimentally the strength and proper proportions of the eyes has been already mentioned, and this applies not only to the usual eyebars and counterbraces used in bridges, but also to many cases where the end fastenings are of a similar nature, such as round solid end connecting rods in machinery, boiler braces, etc. Moreover, sometimes the arrangement is such that the load is more or less eccentric. Beside these cases we have also those where the ends are riveted, and these should be investigated both when the resultant of the load acts along the centre of the bar, and when it does not. There is also opportunity for investigation opened by the various modes in use for fastening the ends of tensile diagonal braces employed in steam boilers. The above is only a partial list of suggestions for experimental study of members of this character.

b **Built up Columns for Buildings and Bridges.** The importance of studying the details, and that of examining the effect of eccentric loading due to the different modes of attaching the floor beams, etc., has already been mentioned, also the determination of the ratio of length to radius of gyration, at which the crushing strength per square inch begins to decrease on account of buckling, and how this is affected by the quality of the steel used, the question as to the relation when the ratio of length to radius of gyration is less than that referred to above, between the crushing strength and the tensile yield point in all kinds and grades of steel.

There is also the question whether the same law may or may not properly be assumed to hold when through overstraining or otherwise a yield point is obtained which is very considerably more than fifty per cent of the ultimate tensile strength, including cases where it approaches it.

Among the causes which result in failure by buckling of the channels in detail, may be mentioned the proportions of the channels or other shapes used and their arrangement.

c **Cast Iron Columns for Building.** The results already obtained show that the principal factors affecting their strength are the flaws, defects, etc., induced by the usual methods of casting such columns, and as it follows that the strength that can be relied upon is the lowest, it would be desirable to study experimentally such methods as would in all cases insure higher results.

Besides the above there is also the question of the effect of eccentric loading.

d When it comes to riveted joints where the members are subjected to either an eccentric or a transverse load, there is a large field opened up for experiment, including end connections, gussets, splices, etc., of plate girders, also the stresses involved in the flanges and flange rivets, the spacing necessary to prevent buckling of the flanges or of the web in detail, and many others.

e The remarks already made upon the tests of reenforced concrete will not be enlarged upon except to say that more light is required regarding the reenforcement of dams and retaining walls and the most effective method of reenforcing arches and domes.

f No further remarks will be made here about the tests of masonry.

g **I Beams.** In regard to the strength of I beams, not only does all that has been said heretofore concerning its variation with that of the chemical and physical properties of the steel, apply here, but also very little has been done by way of studying experimentally the effect of the variation of the properties of the steel in the different parts of the section due to the proportions of the I section and the consequent differences in the degree of work done on the different parts by the rolling.

Another important matter that has hitherto received but little attention is the effect on the modulus of rupture of the proportions of the web and of the flanges respectively, and also of the span employed. Thus there are certain proportions of the web and flanges which for moderate spans will develop on the compressive side the full crushing strength of the metal, and hence result in the greatest modulus of rupture, and the

avoidance of a failure due to the buckling of the flange in detail.

Also for any given section there must be a certain span which if exceeded will result in local buckling and consequently in a lower modulus of rupture.

Of course the failure of I beams of wrought iron or of structural steel is always due either to local buckling or else to the ultimate crushing strength having been reached on the compressive side, and only in the case of exceptionally brittle material is it ever due to a tearing on the tension side.

In view of these facts the experimental determination of the maximum or ultimate transverse strength presents some difficulty (which can be overcome however), and together with some other more or less theoretical reasoning often leads the experimenter to make the chief object of his test the determination of the yield point rather than that of the ultimate, the most common method of ascertaining it being from the load at which the ratio of increase in the deflection becomes noticeably greater.

Moreover when the so-called outside fibre stress at the yield point is obtained in the manner just described, it should be compared with the real yield point of the material as deduced from a tensile test, as it is plain that when a noticeable increase can be observed in the deflections, the real yield point has already been exceeded, since the passing of the outside fibres beyond it would not be sufficient to cause an observable increase in the rate referred to, the latter becoming sufficiently pronounced to be apparent only after a number of the fibres have passed the yield point.

Detailed investigations should be made to find the effect of the form and proportions of section, and for a given section of the span, upon the excess of the yield point as determined from a transverse test, over that determined from a tensile test.

This applies to many cases besides that of the structural I beams used in buildings, and also to many where other sections are used.

h **Repeated and Alternate Stresses.** The work hitherto performed has dealt mostly with the question of the greatest load under which the piece under test can stand without break-

ing, an indefinite number of repetitions or alternations between tension and compression, and more or less has been done to determine the relation of this load to either the elastic limit, or the yield point of the material. Even this work needs to be more extended and to include the cases of heat treated, of alloy steels, and of those whose yield point is very considerably more than one half the ultimate.

One matter of importance which has been but little dealt with is the effect of the repetition or alternation of a certain stress below or above the yield point upon the physical properties of the material. Thus suppose we determine the physical properties of a certain number of specimens chosen from a lot all made from the same heat of steel, and then subject other specimens of the same lot to repeated or alternate stress (beginning by choosing a stress of a supposedly moderate amount) for a certain number of repetitions (say 500,000 or 1,000,000). Then remove one of these pieces from the repeated stress machine and test its physical properties.

If the latter have not undergone any change then it would seem that that number of repetitions of the chosen stress has not caused any change of physical properties, and with the next piece we should increase either the load or the number of repetitions or both. Another important matter is to make an experimental study of the effects of keyways, set screws, etc., upon the lowering of the allowable repeated stress.

Such tests as these should be also made upon heat treated and alloy steels, and upon steels having exceptionally high yield points as compared with the ultimate.

Lately attempts are being made by several to compare the resistance to repeated stress of different specimens of steel by subjecting them to repeated or alternate stress of such magnitude that they can be broken in a short time, as an hour or less.

The question of what can be fairly concluded from such tests is one which is at the present time under discussion, and regarding which more evidence is needed before the proper answer can be found.

i The whole subject of impact has been but little investigated experimentally, and what little has been done has had mostly to do with breaking. Though it is true that impact

tests of certain kinds are called for in specifications for axles, rails, wheels, etc., and that a very large number of this class of tests are constantly being made, nevertheless the weight and drop to be employed have been fixed upon more or less arbitrarily in the light of past experience, and a study of the effect of impact upon the physical properties of the material, and the question of the proper weight and drop for proof tests have not yet received the requisite attention in an experimental way.

There has arisen within recent years the recognition for the need of proof tests, that is, such as will reveal defects in the member but which, if it is sound and of proper quality, will not strain any fibre beyond the yield point, that is tests which can be applied to each member if desired, or to a certain number of selected members, but will not injure them for service if they are not defective, but which at the same time are of sufficient severity to reveal any cracks, even incipient ones, any undue segregation, imperfect welding or other defects if they exist.

Such tests are becoming more and more necessary now that heat treated and alloy steels are coming into extensive use.

Indeed the whole subject of proof tests by impact is in a very embryonic state, and precise experimental work is very much needed.

Again, looking at the matter of impact tests from a more fundamental and theoretical point of view, we may observe that very little has been done heretofore that will enable us to ascertain, in the case of any given impact testing machine, what per cent of the energy of the falling weight or of the pendulum is actually used up in distorting the specimen, and how much in distorting the machine, its foundation, and the surrounding ground.

The finer work, such as the securing of a correct stress strain diagram which will show the actual deformation of the specimen during the time when the ram is in contact with it, has been but little investigated.

Though perhaps hardly to be classed as full size tests, it may be observed that it is only in recent years that attempts have been made to study the value of tests by impact of nicked bars, and to determine what such tests show regarding the

quality of the bars, and how they can be so standardized as to furnish useful, and at least comparative results of the quality of the material.

Taking up, next, members belonging to the second division described above, the following may be cited as examples of certain cases where full size tests are needed, and where the results available in print are few.

1 **Steam Boilers and other Pressure Vessels, as Pulp Digesters, etc.**

a The effect of the ends upon the distribution of the stresses in the shells, both in the case of unstayed ends and where different methods of staying are employed.

b The distribution of the stresses among the stays and in the stayed surfaces with staying of different kinds, and those developed by the various modes of fastening of the stays.

c The stresses in flat ends where no stays are used, and where they are.

d The determination of the load on each of the stays, and the bending moments in the plate where the surfaces into which the stays are screwed are not parallel, as in the case of a radial stay boiler.

e The stresses in spherical ends, both at the centre and at the flanged portions where they are attached to the shell.

f The effect upon the stresses of heating a portion of the shell, as in the case of the horizontal multitubular boiler and others.

In all the above some form of strain gauge would be a valuable tool for use. The strains should be measured in more than one direction at any one point, as a rule in two at right angles to each other. Such tests should be made, not merely upon riveted cylindrical vessels but also upon those welded throughout and having spherical ends, as in a number of pulp digesters.

Observe also that all this should be done upon full size boilers and digesters, and not upon small models.

2 **Hooks, Links, Chains, Machine Frames, etc.** More or less work has been performed along these lines, but the information hitherto obtained is far from adequate for our needs. Thus in the case of hooks, we have first the common or older theory,

and at least one other, the mathematics of which is based upon the Theory of Elasticity.

In seeking experimental confirmation of these or other theories, difficulties are encountered especially with ductile metal.

In such cases not much satisfaction can be obtained from experiments upon the breaking loads, and in consequence of this fact some have tried to base their conclusions upon an experimental determination of the yield point, but they have not always been careful to observe that the fibre stress corresponding to the yield point load as determined from deflection measurements can not be relied upon to give the true yield point of the material as determined by tensile tests, and that the difference between the two varies with a number of circumstances, one of which is the form of the section.

In the case of links the amount of experimental evidence that will determine the distribution of stresses is small, although the breaking strength of chains has often been tested, and many results are available.

These however do not enable us to deal with the various forms and proportions of links, rings, and straps that are employed in practice. A parallel is to be found in various parts of machine frames, especially punching machines and riveters.

3 **Connecting Rods and Links in Running Machinery.** In these we most commonly have to reckon with the tensile, the compressive, and the transverse stresses in the body of the rod of link, and often with a combination of two of them.

Observe moreover that generally the rods are not built up as with bridge and building columns, and here we have to deal with solid sections, and also with a grade of steel differing very much from structural steel.

Therefore the value of length to radius of gyration at which the crushing strength begins to decrease on account of buckling is very much higher than it is in built up columns.

Besides this, much needs to be done in regard to the strength of the end fastenings, and also the effect of using the recently introduced steels already referred to.

4 **Pistons.** While a little has been done, much more is required with pistons of cast iron, of wrought iron, or different

kinds of steel, also upon different forms, as box pistons, spider and follower pistons, pistons largely in the form of flat plates with a ring at the outside, and those in the form of a cone.

Also the effect of using radial or other ribs in box or spider and follower pistons.

What has been done hitherto has led to empirical and semi-empirical methods of calculation rather than to methods based upon the Theory of Elasticity.

5 **Cylinder Heads.** While those which are really flat plates can be treated by methods depending upon the Theory of Elasticity, and while constants for use as determined by experiment are available, by far the majority of cylinder heads are not amenable to such treatment, as their forms are very complicated, involving not merely plates, but also ribs and other projections, while they are often conical or of a curved form. While some good results have been secured, leading to a semi-empirical treatment, much more should be done by way of experiment before we have sufficient information to cover all cases. It is probable that the resulting formulae will have to be at least partly empirical, and in this event the tests will have to include practical examples of all sorts and cover a sufficiently wide range, as such empirical and semi-empirical formulae do not furnish us the means of passing from certain sizes and proportions to others differing very much from them.

Many other sorts of full size tests might be mentioned, some of which would involve the use of very powerful testing machines and special apparatus, as for instance testing the entire chord of a bridge, the entire frame of a machine, trusses of timber including framing joints, whether made by bolts, notches or shoes of various patterns bolted on, etc.

Of course such a set of tests as those outlined in this paper is not possible for any one laboratory, both because their equipment and the time available would be inadequate. When the question arises as to how and where portions of the work can be accomplished, the possibilities that will readily occur to any one are the following, viz.:

1 Engineering Schools.

2 Experiment Stations, whether connected with a school or not.

3 Industrial Works.

4 Government Laboratories and Bureaus.

1 The Schools are limited in their resources and equipment and in the number of persons available who are competent to carry on such investigations.

Therefore it is only in the larger ones that much work of this character can be performed. Besides this the schools are established for teaching the students, and only such amount of investigation is feasible as is consistent with the carrying on of the work of teaching.

2 Experiment Stations. The amount that can be done in these establishments depends upon the equipment and resources.

3 Industrial Works. Sometimes full size tests are made by such concerns, but this as a rule can only be done when they are specially and commercially interested in the results.

4 Government Laboratories and Bureaus. How much they can do depends upon the amount of money that the lawmakers deem it wise to spend for equipment, for a competent personnel, and for carrying on the work.

In view of the above it is plain that by cooperation, not only will unnecessary duplication be avoided, but also where it is desirable that certain tests should be made by independent observers and the results compared, it can be done.

By a systematic cooperation and communication between the agencies that are doing work of this character, the greatest good can be accomplished.

DISCUSSION

Mr. Wilson. **Mr. R. M. Wilson*** said that the question of the buckling of flanges, and the effect on the modulus of rupture of the relative proportions of the web and flanges of I-beams, have been referred to on page 638, and one would expect the same questions to have some bearing on the constants used for the straight-line formula commonly used in column design.

In the design of columns, formulae have been developed and used, based on the moment of inertia of the section and without regard to the manner in which the section has been built up.

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Formulae have also been developed for the purpose of calculating the bending and horizontal shear in columns, based on given assumptions. Mr. Wilson.

It appears to Mr. Wilson that, in specifying the straight-line formula, the radius of gyration of the section is considered and no account is taken of the interrelation of the parts forming the cross-section of the strut, although experiments have been made to show the results obtained, due to the buckling of individual members.

One of the writers in Engineering News, January 1, 1914, when discussing the probable causes of failure of a long H column, suggested the buckling of the flanges as a possible cause of the trouble, and when rebuilt, the column was stiffened at the edge of the flanges by using channels, instead of flat plates.

Mr. Wilson believes that some light may be thrown on this question from an inspection of the tests to destruction of full-sized steel columns, as exhibited by the United States Bureau of Standards, Liberal Arts Building, Panama-Pacific International Exposition.

These tests tend to show that sections having unsupported flanges fail at lower unit stress than those having stiffened flanges. He would very much like to see the results of tests on the type of H columns which have been used recently in the lower stories of tall buildings, and to know whether there should be any modification of the constants of the usual straight-line formula for such cases. In these columns, the relative proportions of web and flange areas are very different from those usually employed, and the speaker would expect them to fail with a lower unit stress than the more common channel section.

He is of opinion that if the above type of H column is used, either the stress usually specified for struts should be decreased, or the outer edges of the flanges stiffened up with angles, and, in some cases, also lattice bracing or batten plates.

Mr. Frank S. M. Harris,† Assoc. M. Am. Soc. C. E., said that in listening to Prof. Lanza's paper on "Testing Full Sized Members", the writer was impressed with the broad general outline of tests suggested, and the vast amount of data to be derived therefrom, particularly as regards built-up members of the larger structures, as bridge chords and the like. Mr. Harris.

The obvious drawback in this particular branch of full-size testing has been the magnitude and expense of the machine required for the testing to failure of such tremendous specimens, as well as the practical difficulties in the accurate calibration and control of the loading. The reliability of the calibration of large testing machines of the hydraulic type has, until recently, been so seriously doubted as to materially discourage the introduction of this type of machine, which, of all others, could be most readily and economically adapted to large capacity work.

The tension chord of the large structure has fared better in this regard than has the compression chord. This is due to its being most frequently composed of a system of eye-bars, the testing of any unit of

† Oakland, Calif.

Mr. Harris. which, individually, gives the true strength of all, collectively. This observation, of course, does not apply to built-up tension members, full-size tests of which the writer believes have never been made.

Great uncertainty has always attended the construction of large built-up compression members, due to the many specific factors which enter into their design, such as the degree of fixedness of the ends, the slenderness ratio, the resistance of the plates to buckling and the strength of the lattice to take the transverse stresses resulting from the axial loading. In spite of these gross uncertainties and the urgency for their empirical determination, full-size tests of the compression members of the most important structures are precluded by the tremendous loading required for their ultimate failure. The heaviest chord sections of the dimensions of the Blackwell's Island or the Quebec bridges would require an axial load of from 15,000 to 20,000 tons, while the capacity of the Phoenixville machine is but 1,200 tons.

Mr. Gustav Lindenthal and others have suggested the using of quarry walls as abutments between which to test such full-size members, loads up to 25,000 tons being applied by means of either an hydraulic cylinder acting directly at one end, or by some form of toggle joint at the center between two such members. These proposals have never met with favor.

The practical solution of this difficulty, which at once suggests itself, lies in the testing of scale models of the members, reduced only sufficiently to come within the capacity of existing testing machines. It is the necessity of such testing, and the belief of its value, that prompt the discussion.

The primary object in the making of full-size tests is not data regarding the physical characteristics of the material of the specimen but, rather, information regarding the conformation and distribution of the material in the cross-section so as to cause it to work together as a whole. Such questions would be answered qualitatively, at least, if not quantitatively, by the testing of properly made scale models. Among the more urgent of these problems may be mentioned the following:

(a) The transverse stress in terms of the axial stress on the member, necessary in the design of the lacing.

(b) The distribution of the stress in the cross-section. This distribution depends largely upon the end conditions of the member, and while neither the reduced nor the full-size test can truly reproduce the end conditions of service, they may be studied quite as accurately in the model as in the large member.

(c) The point where the slenderness ratio begins to hold, and a suitable reduction factor for use in column formulae. Knowing the physical properties of an unfamiliar alloy steel or of a high-carbon steel, the testing of a scale model should give very definite results.

As pointed out by Prof. Lanza, small sections, such as would compose a test model, receive relatively more manipulation in the rolls than do the larger sections in the make-up of the full-size member, with a correspondingly higher elastic limit and ultimate strength. If thought

Mr. Harris.

advisable, this error may be guarded against by the reduction of the section of those parts in proportion to the rise in their ultimate resistance. A case in point would be the small angles used for the lacing of a model strut, which could be smaller than the scale dimension to make them comparable with the elements of the main section which would be of lower ultimate resistance. All details of fabrication should be reproduced in so far as practicable, such as sub-punching, reaming, drilling and the sequence of riveting. This will, in a measure, overcome the tendency to take unusual care in the fabrication of the model, which would result in its being tested under "idealized" conditions.

The largest compression tests thus far undertaken were of models of the compression chords of the Quebec bridge (1907), one of the ill-fated member which first failed in the original design, and one of the revised design of the same member. One of these models was at one-third linear scale and the other at one-fourth linear scale, but even these reduced members required nearly the full capacity of the Phoenixville machine for ultimate loading. The results of these tests, which were satisfactory in every particular, now that the calibration of the machine is determined, are to be found in detail in the report of Professor Wm. Burr, M. Am. Soc. C. E.

The writer is in full accord with Prof. Lanza as to the desirability of full-size testing wherever practicable, but lacking facilities, the testing of the model, fabricated as nearly as may be under "full-size" conditions, will at least contribute materially to the meagre information at hand.

Paper No. 114½

NOTES ON CORROSION IN IRON AND STEEL STRUCTURES.

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London, England

USE OF WROUGHT IRON VERSUS STEEL.

With special reference to the use of wrought iron versus steel for purposes where either might answer the structural requirements, some miscellaneous notes may be of interest.

In coal-carrying wagons I am told that the corrosion of the standard steel sections of the underframe amounted to ⅛ in. in fifteen or sixteen years. The main members became so weakened, in consequence, that when the trucks received a sharp impact, the parts were knocked out of shape and the life of the wagon immediately came to an end. It has further been stated that in railway experience the corrosion in the case of iron wagon fittings was from 40 to 50 percent less than in the case of steel.

It was common experience that engineers who had employed wrought iron in recent practice for locomotive fire-boxes had done far better than those who had used steel.

In the case of bolts, 90 percent of the bolts and nuts made in the United Kingdom were made from wrought iron, although bars may be rolled for this trade made from a mixture of iron and steel of non-homogeneity for the sake of cheapness.

Clearly, then, the revival of wrought iron for general work will be closely dependent upon a reliable supply of wrought iron produced from the ore.

For the skins of gates for dock entrances and locks, best wrought-iron plates are preferred by engineers, the idea being that the iron is less liable to corrosion than steel, this being

especially an important consideration in dealing with under-water parts rarely accessible for cleaning and painting.

Of course in the case of graving-dock gates which can be got at for painting, etc., as often as necessary, there does not exist the same reason to prefer wrought iron, and, consequently, mild steel may be adopted.

CORROSION OF CAST IRON.

Hard and crystalline iron, in which the carbon is chemically combined with the metal, is found to be less oxidable than ductile and fibrous iron. Cast iron is not so readily oxidized in moist air as wrought iron, but when influenced by impurities in water, corrosion becomes accelerated by the presence of decomposing organic matter or free acids. Cast-iron water pipes laid in gas-soaked soil, or in ashes in which there is sulphur, or in certain clays in the London area, have been found to deteriorate rapidly, the iron becoming in a few years converted into a chemical union of carbon and iron known as ''plumbago'', so that it could easily be cut with a pocket knife, while other cast-iron pipes laid in suitable soil have lasted and been in use over sixty years, the metal appearing as good as when the pipes were laid. Both the East London Water Works Company and the New River Company commenced to use cast-iron pipes about the year 1808, and in the year 1816 iron mains were said to be charged regularly throughout the night, the higher pressures required rendering the old wooden pipes, which had hitherto done good service, less suitable than the iron pipes. Steel pipes have also been introduced, but when brought into connection with water of a certain quality, the electrolytic action is proportional to the time during which the current acts; and except over ground where transport is distant and difficult, cast-iron mains cast vertically are preferred.

ELECTROLYSIS.

Electrolysis is the splitting up of compounds into their chemical constituents by the action of an electric current passing through them.

The simplest form of electrolysis is the decomposition of water into oxygen and hydrogen by the action of an electric current. Dr. Guillot, of Paris, presented a paper at the recent

autumn meeting of the Iron and Steel Institute in which he showed that by the manufacture of electrolytic iron, a comparatively pure material was obtained which was specially adaptable for use in the construction of electrical machinery but was brittle and hard, due probably to the occluded hydrogen as a result of the process, but improved by annealing for portions subject to compression or bending.

This iron is stated to corrode very easily, probably on account of its content of iron chloride.

The presence of two dissimilar metals in a corrosive atmosphere acting upon both of them, may bring them into electrical contact and the whole of the corrosive influence may act on one of the metals without reducing the section of the other.

Corrosion may also be accelerated by the contact of iron with any metal which is electro-negative relative to the iron, or, in other words, has less affinity for oxygen or with the rust of the iron itself. If two portions of a structure of iron are in different conditions so that one has less affinity for oxygen than the other, the contact of the former may under certain conditions make the latter oxidize rapidly.

In the case of an iron boat in which steel rivets were employed in the repairs of an iron plate on the top of tanks under a boiler, the steel oxidized at the expense of the iron, a galvanic couple being formed. In a steel boat in which iron rivets were used in repairing the deck, these became slack, and had to be replaced by steel. The softer material may have been sheared by the harder steel, but the wearing away was doubtless accelerated by galvanic action. The importance of thoroughly removing any scale from the steel surface is seen in the fact that galvanic action may be set up between the adherent scale and the adjacent bare portion of the steel surface, the latter suffering locally and pitting taking place; but it is more difficult to remove scale from steel by chipping or scraping than it is to remove scale from iron.

DECOMPOSITION OF CAST-IRON PIPES.

The theory of decomposition of cast-iron water pipes in contact with the ground has varied considerably during the last few years. At one time it was thought that the moisture in the ground was split up into its elements, oxygen and hydrogen, and

that the iron was acted upon by the liberated oxygen, and gradually disappeared in the form of an oxide; but later investigations seem to indicate that the action is that of a transposition of the iron molecules, the action being very similar to that which takes place in an electro-plating bath, where the particles of silver are transposed from the silver plate to the articles immersed.

In the case of the water pipes in the ground, the particles after disintegration tend to travel towards the electric current, which, in many cases, is provided by a tram rail, but, of course, it is very unlikely that many of them actually reach this destination, as in passing through the ground they come into contact with the nitrates and chlorates, and are converted into ferric salts.

In any case, the essential factor appears to be the presence of water, and the action cannot take place in perfectly dry ground. The water is decomposed and the hydrogen liberated.

Steel is more easily oxidized than wrought iron and far more easily than cast iron. Generally speaking, though, the oxidation of steel is faster than that of iron, it is more uniform, and the corrosion of both iron and steel is more rapid when partly wet and partly dry than when wholly immersed in water or wholly exposed to the air.

CORROSION OF WROUGHT IRON IN SEA WATER.

In examining the viaduct of the Prince of Wales Pier at Dover, where wrought-iron deck girders had a clear headway of 15 feet above the level of high-water spring tides, the under sides and bearings of these girders exhibited the most corrosion. Upon the supporting piles, the maximum corrosion occurred at high-water level. Below this level the shell incrustation on portions subject to immersion seemed to form a protection, because if it were removed by hand the black varnish covering underneath, which had been previously applied, appeared to be undisturbed. The under deck parts and the troughing in the viaduct are examined, cleaned, and black-varnished periodically, while all portions above deck are repainted with "Perfectum" paint, a material that has been found to give satisfaction for a durable covering, free from cracks.

Nothing appears to equal black varnish as a protection against corrosion for portions of iron or steel structures immersed in tidal waters when it is laid upon the hard metal, preferably applied upon a dry surface; but it is necessary to prepare the surface to receive the varnish by cleaning off any scale with a wire brush, and to take care that the varnish employed is black varnish and not "thinnings".

GALVANIZED SHEETS.

Iron sheets coated with zinc, known as galvanized iron have been, when corrugated, more used for roofs than any other metallic covering. In pure air these are long-enduring, if the quality be good; but the thinner sheets may be easily pierced and a surface crack or scratch of the zinc allows the commencement of rust, the crack developing into a serious rent, when the sheets become worthless. In regard to endurance against rust, good iron, well galvanized, will last longer than thicker sheets of inferior quality, but iron is not an imperishable material and its life depends upon the agencies to which the metal becomes exposed.

Where it is necessary to place iron in contact with zinc, it should be galvanized, but where otherwise, thicker sections of iron will allow for oxidation and scaling during a reasonable period. A galvanized-iron roof may last for 20 to 25 years, but will during this time need repair. Zinc being more electropositive than iron, and zinc being particularly sensitive to the action of chlorides and sulphates, it follows that a battery constructed of zinc and iron with the presence of sea water as a fluid would cause the zinc to be attacked just as it would in an ordinary battery; but since the difference between iron and zinc is not great, the battery would be feeble, and the amount of zinc consumed would be, consequently, such that the zinc would protect the iron as long as the covering lasted. It is thought by some that the tensile strength of the iron becomes diminished by treatment in a bath of molten zinc after corrugation, but this is of comparatively little value, as the life of the covering will have lasted a reasonable time; while in the case of bars of greater section, chains and small anchors, the effect is less appreciable, as thin iron oxidizes more rapidly than thick iron.

In 1861 the dome of the Borough Market, London Bridge, was covered with corrugated iron, but in 1891 this was found to be so corroded and riddled that it was removed by the writer when he reconstructed the present roof over the market, and relieved the roof from the vibration of the adjacent railway.

PROTECTIVE COVERINGS.

The value of paint as a protective covering depends mainly upon its own power to adhere chemically to a surface and to resist decay. In dealing with its preparation, the quality of the bases or pigments employed demands attention, as no treatment can render an adulterated pigment serviceable. Water, oils and spirits of turpentine are all valuable constituents, but the oil employed for mixing needs attention both in its quality as well as in subsequent application. It is a fallacy to assume that if a pigment is good, carelessness in mixing and applying can not be prejudicial, as much disregard of this precaution may result in peeling off, blistering or perishing. Consideration of driers, to act as carriers of oxygen to the oil in order to aid the oil to solidify, is an expedient in emergency; and when the work is exposed to the sun, the use of turpentine prevents the paint from blistering. Covering power is a necessary element in the economical use of paint and is ultimately connected with fineness of grinding and uniformity of execution. The examination of commercial paint must be left in the hands of a trained chemist, but much anxiety may be saved by employing a paint made by a respectable house of manufacturers.

The paint trade becomes a spurious one with second-hand firms who may seek orders by submitting low prices, because it is possible to give a large percentage of adulteration. For instance, white paint may be commercially supplied without any white lead in it, ''barytes'', known sometimes as ''dutch lead'' in the market, being substituted. This produces an ''earth white'' as ponderous as chalk, and it is known that sulphate of baryta absorbs very little oil.

Machinery is acknowledged to grind colour finer than when the material is ground by hand and the result is found to be improved when ground in oil, as each particle is then completely covered with oil. Powdered stuff subsequently mixed with oil

will float and prove to be only mechanically mixed, so that the painted surface will not appear as smooth as when more thoroughly mixed by the process of grinding in approved oil. Ordinary paint for indoor work is usually prepared with raw linseed oil, while that used out of doors contains boiled oil, as its rapidity in drying contributes to a protective covering.

The surface of the metal to be protected must, of course, be prepared by being properly cleaned from rust and scale and the paint should not appear to dry quicker than is required for a thorough, superficial covering. Experience teaches that the quicker the drying, the shorter time will the covering last. While water dries by evaporation, and leaves nothing behind it when a surface dries, linseed oil dries by absorption of oxygen and becomes changed into a substance, so that the drying property of linseed oil must not be confounded with the drying of a substance wet with water. The object of extra coats of paint, applied one by one, when the surface to be treated is dry, is to ensure the covering being non-porous, since air and moisture are sure to penetrate through any cracks in the dried oil, such minute accrescences being mainly the result of the difference in the rate of expansion and contraction between the metal to be protected and the paint covering it. The difficulty of finding a form of paint absolutely impervious to moisture and to gas has led to numerous patent combinations more attractive in their name than in their manufacture. Oxides of iron paints are deemed preferable to lead paints for iron and steel work generally, as in the case of lead covering, a galvanic action may be set up between the lead and the iron, particularly if lead is present as metallic lead. The carbonate of lead could not produce it alone. There is also a tendency of red-lead paint to solidify if not used soon after it is made, which may be due to the combination of the lead with any acid in the oil, and rust may manifest itself in spots, though such paint adheres with firmness, when moisture is carried through a film of a first coat of red-lead covering. Moreover, when red lead is used as a priming or first coat, if the paint is mixed with rule of thumb proportions, it may not assimilate with future coats of paint; so that notwithstanding the provision of a covering from the air, the red lead may oxidize with the iron, thus forming rust, which, although minute,

would be sufficient to detach the paint of a second or third coat. In most cases rust, if it occurs, forms itself beneath a paint covering, and, consequently, when the rust is thrown off, the paint comes off with it. Hence, as an agent to prolong the life of an iron or steel structure, expert supervision of application is as important as the manufacture of the paint employed for its protection.

There has never yet been discovered any coating to preserve iron against atmospheric effects, except in so far as any adopted covering may prove impermeable. Portland cement grout or concrete of approved quality applied so as to leave no vacuities is considered effective.

The importance of the removal of all dirt, moisture, rust and scale from an iron or steel surface, as a precautionary measure to the application of a preservative covering, cannot be overrated, especially in the case of a steel surface when the rust consists of magnetic oxide, which is electrically negative to steel, and, therefore, a contributory element to corrosion.

Pure and dry oxygen is found by experiment not to determine the oxidation of iron.

Even moist oxygen has only a feeble action, while dry or moist pure carbonic acid has no action; but oxygen containing traces of carbonic acid will create protoxide of iron, with a tendency to carbonate of the same oxide accompanied by a mixture of saline oxide and hydrate of sesquioxide.

Oxygen and carbonic acid are acknowledged to be necessary to produce oxide of iron in the presence of moisture.

The practice of burying steel stanchions and girders in walls, where they are imperfectly protected from climatic influences or from moisture percolating through the brickwork, is, to say the least, an inexpedient proceeding.

When embedded in brickwork, any decomposition of the latter may introduce salts of corrosive tendency, and with a concrete covering there should be no porosity in order to insure air- and water-tightness for durability.

In a paper read by the writer at a meeting of The British Association, held in 1900 at Bradford, Yorkshire, dealing with expanded metal as applied to reinforced concrete, the Author reviewed the general conditions of the combination of steel and

concrete, and in a discussion at The Institution of Civil Engineers in 1914 upon a paper by Mr. Wentworth Shields upon Dockwork at Southampton, the Author stated that "if the concrete were at all porous, the use of sea water might affect the concrete in a far more prejudical manner than by causing rusting of the surface of the steel reinforcement", as in the case of Portland cement concrete mixed with sea water.